CONNECTICUT RIVER BASIN

CANTON, CONNECTICUT

COLLINS COMPANY UPPER DAM
CT 00674

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

JULY, 1979

407-440

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Connecticut River Basin
Canton, Connecticut

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam facility consists of a 325 foot long masonry spillway and a 335 foot l earthen embankment which span the Farmington River and are referred to as the dam. Based upon the visual inspection at the site, past performance of the dan and existing data, this dam is judged to be in good condition. Based upon the size classification (intermediate) and hazard classification (significant) the test flood will be equivalent to ½ the PMF.

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	COLLINS COMPANY UPPER DAM
Inventory Number:	CT-00674
State Located:	CONNECTICUT
County Located:	HARTFORD
Town Located:	CANTON
Stream:	FARMINGTON RIVER
Owner:	STATE OF CONNECTICUT
Date of Inspection:	APRIL 26, 1979
Inspection Team:	PETER M. HEYNEN, P.E.
	CALVIN GOLDSMITH
	THEODORE STEVENS
	GONZALO CASTRO, P.E.
	CHARLES OSGOOD

The dam facility consists of a 325 foot long masonry spillway and a 335 foot long earthen embankment which span the Farmington River and are referred to as the main dam. The top of the earthen embankment is approximately 32 feet above the downstream riverbed. There are two other dams adjacent and perpendicular to the downstream face of the main dam. The 100 foot long forebay dam impounds the forebay, the body of water downstream of the left end of the dam which is utilized for industrial purposes by the adjacent factory. The wing dam, a 200 foot long dam consisting of a 160 foot long spillway and a 40 foot long abutment located at the right end of the main dam, was originally intended to route flow to an adjacent powerhouse, which is no longer operational. Outflow from the project is over the three dam spillways and through eight sluice gates on the main dam, one mid-level and three low level sluices, as well as the closed intakes to the powerhouse, on the wing dam, and from a 42 inch square sluice and a 60 inch diameter pipe on the forebay dam.

Based upon the visual inspection at the site, past performance of the dam, and existing data, this dam is judged to be in good condition. No evidence of instability was observed in the main dam or the two appurtenant dams.

Based upon the size (intermediate) and hazard classification (significant) in accordance with Corps of Engineers Guidelines, the test flood will be equivalent to one-half the Probable Maximum Flood (PMF). Peak inflow to the dam impoundment is 83,000 cubic feet per second (cfs); peak outflow is

83,000 cfs with the dam overtopped 4 feet. Based upon two existing flood plain reports and our computations, the spillway capacity is 55,000 cfs, which is equivalent to approximately 66% of the routed test flood outflow.

It is recommended that the owner initiate further research by a qualified registered engineer to determine if a detailed evaluation of the capability of the dam to resist overturning, based upon uplift pressures to be measured and accurate determinations of the configuration of the dam foundation, is warranted.

The engineer should also examine the downstream face of the dam structures during no-flow conditions, and make any needed repair or renovation recommendations based upon his field observations.

The above recommendations and any remedial measures, all of which are discussed in Section 7, should be implemented by the owner within two years of his receipt of this report.

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

Edgar B. Vinal, Jr., P.E.
Senior Vice President
Cahn Engineers, Inc.

This Phase I Inspection Report on Collins Company Upper Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

SAUL C. COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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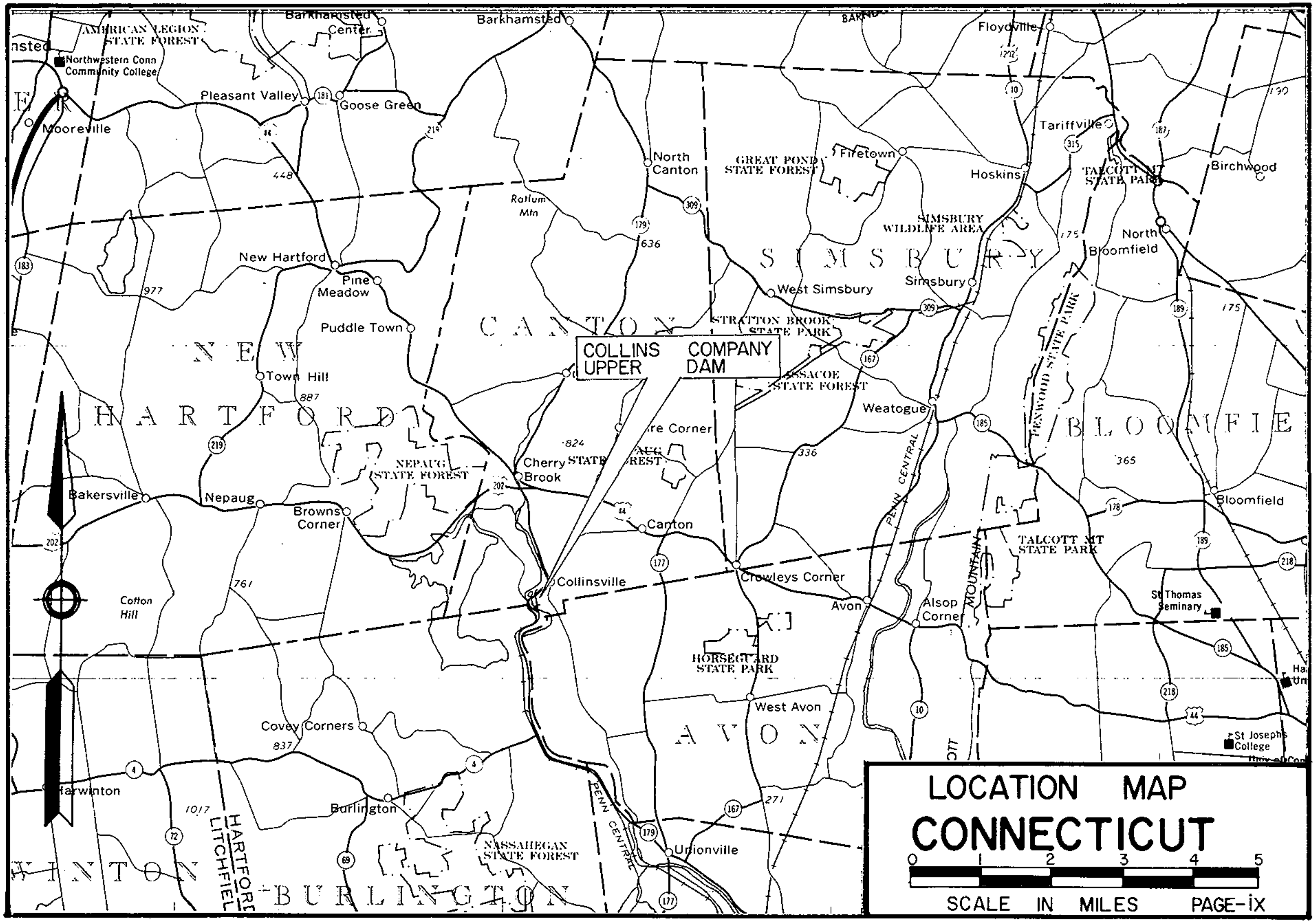
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		FARMINGTON RIVER	CONNECTICUT	CE # 27 595 KB PAGE viii


COLLINS COMPANY UPPER DAM
NEW HARTFORD
CANTON
SIMSBURY
BLOOMFIE
AVON
BURLINGTON
Barkhamsted Center
Barkhamsted
Floydville
AMERICAN LEGION STATE FOREST
Northwestern Conn Community College
Pleasant Valley
Goose Green
Mooreville
North Canton
GREAT POND STATE FOREST
Firetown
Tariffville
Hoskins
TALCOTT MT STATE PARK
Birchwood
Rattlesnake Mtn
SIMSBURY WILDLIFE AREA
North Bloomfield
New Hartford
Pine Meadow
West Simsbury
Simsbury
Puddle Town
STRATTON BROOK STATE PARK
Town Hill
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NEPAUG STATE FOREST
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Nepaug
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Canton
Bloomfield
PENN CENTRAL
Collinsville
Crowleys Corner
Avon
Alsop Corner
St Thomas Seminary
Cotton Hill
HORSEGUARD STATE PARK
West Avon
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PHASE I INSPECTION REPORT

COLLINS COMPANY UPPER DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of November 28, 1978 from Max B. Scheider, Colonel, Corps of Engineers. Contract No. DACW 33-79-C-0014 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on the Farmington River in an urban area of the Town of Canton, County of Hartford, State of Connecticut. The dam is shown on the Collinsville USGS Quadrangle Map having coordinates latitude N 41°48.6' and longitude W 72°55.6'.

b. Description of Dam and Appurtenances

As shown on Sheet B-1, the main dam is approximately 32 feet above the Farmington River, consisting of a masonry spillway, approximately 325 feet long, to the right of an approximately 335 foot long earthen embankment. The earthen embankment at the left end of the main dam is faced on its upstream side by a vertical masonry retaining wall which has been partially reconstructed with concrete. Towards the center of the embankment is a 62 foot long concrete bulkhead for eight sluice gates. The downstream side of the 13 foot wide bulkhead section consists of a vertical masonry retaining wall. Flow from the gates passes beneath a single span highway bridge and into a forebay for the adjacent factory buildings. The forebay is impounded by a 110 foot long masonry dam similar in construction to the main dam spillway. At the right end of the forebay dam are a 42 inch square sluice and a 60 inch diameter low level outlet pipe. Connecticut Route 179 runs along the top of the embankment and the single span bridge above the inlet to the forebay, and then along a four span bridge across the Farmington River immediately downstream of the main spillway. Near the right end of and perpendicular to the spillway section is a 200 foot long concrete wing dam or "diversion dam", which directs flow downstream to an abandoned brick and concrete powerhouse. At the upstream end of the powerhouse intake channel is a 20 foot long notch in the main spillway stepped to a maximum depth of 3 feet below the spillway crest (Sheet B-1). The wing dam has a 160 foot long spillway at an elevation one foot lower than the main dam spillway, and a 40 foot long right abutment housing one mid-level and three low level outlets. Intake to the powerhouse is through a timber slide gate protected by trash racks. The powerhouse tailrace channel is confined on the left by a low concrete training wall and on the right by a dry laid stone retaining wall. The main dam spillway, the concrete wing dam spillway and the masonry forebay dam spillway are all able to accommodate flashboards.

c. Size Classification - (INTERMEDIATE) - The dam impounds 1400 acre - feet of water with the water level at the top of the dam, which at elevation 299.8, is approximately 32 feet above the downstream riverbed. According to the Recommended Guidelines, this dam is classified as intermediate in size.

d. Hazard Classification - Downstream hazard was analyzed with the upstream water level 1) at the spillway crest elevation, and 2) at the top of dam elevation. There are 5 houses downstream of the dam which could be affected by a failure of the dam. A breach of the dam with the water level at the spillway crest would cause a rise in the downstream water level which would be 7 feet below the 5 houses, and therefore resulting in no hazard. A breach of the dam with the water level at the top of the dam would cause a rise in the water level to 5 feet above the floor elevation of the 5 houses, however the downstream water level would have already been 1 foot above the floor elevation prior to the breach causing any residents to have been evacuated already when the breach occurs. Therefore, a breach of the dam with the water level at the top of the dam would create no additional hazard downstream.

The significant hazard classification is due to the hazard to recreational users of the river downstream of the dam, who would be endangered by a failure of the dam with the water level at the spillway crest elevation.

e. Ownership - State of Connecticut
Department of Environmental Protection
Region 1 Headquarters
Pleasant Valley, CT 06063
Mr. Anthony Cantelle
(203) 379-0771

The dam was originally constructed and owned by the Collins Company. It was sold to the Hartford Electric Light Company in 1966 which shortly thereafter turned ownership over to the State of Connecticut.

f. Operator - The gates in the bulkhead of the earth embankment of the main dam and those in the forebay dam are operated by the Perry Company which presently occupies some of the old Collins Company buildings adjacent to the forebay.

Mr. Thomas Perry
The T. M. Perry Company
Canton Center, Conn. 06020
693-8356

The four gates located in the concrete wing dam adjacent to the powerhouse are operated by a local water ski club, which has authority to do so as well as to install and maintain flashboards on the dam.

Farmington River Water Ski Club
Mr. Thomas Hinman
Himman's Nursery
River Road
Canton, Conn. 06019
693-0147

g. Purpose of Dam - The dam was used for many years to generate hydroelectric power. The power generation facilities are now non-operational, however, a feasibility study to restore them has recently been undertaken. The impoundment above the main dam is used for recreational purposes and the forebay is used for industrial purposes and as a supply of water for fire-fighting.

h. Design and Construction History - The following information is believed to be accurate based on the plans and correspondence available. The present masonry spillway was constructed in 1837 immediately downstream of an original timber dam which was left in place. The dam was raised two feet in 1849 in order to provide additional water storage. The powerhouse and concrete wing dam were completed in the late 1920's. The highway bridge at the site was constructed during the late 1950's, replacing an earlier bridge which was destroyed by the flood of August, 1955.

i. Normal Operational Procedures - The Perry Company, which occupies the old Collins Company buildings, operates the gates at the left end of the main dam and those in the forebay dam as needed in order to maintain the forebay at full capacity. This normally entails keeping some of the gates to the forebay inlet partially open and, during low flows, keeping the gates in the forebay dam closed. The Perry Company also maintains flashboards on the forebay dam to provide additional storage in the forebay. The forebay dam flashboards are effective in maintaining higher stages in the forebay only when the main spillway and wing dam spillway flashboards are in place.

The local water skiing club maintains flashboards on the main spillway and wing dam spillway. They open the gates in the wing dam to lower the upstream water level in the spring to facilitate installation of flashboards. The club replaces the flashboards yearly as they are regularly broken away by floating ice and/or high springtime flows. During periods of low flow, the gates are closed to impound as much water as possible for recreational purposes.

The powerhouse is not presently in use, thus there are no operational procedures followed for its gates, which are presently in the closed position.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area is 359 square miles of rolling to mountainous terrain of which 140 square miles are (partially) controlled by 3 upstream flood control reservoirs; Colebrook River Dam, Mad River Dam and Sucker Brook Dam. Four other dams are also located in the watershed. These are Highland Lake Dam, Saville Dam, Richard's Corner Dam, and Nepaug Reservoir Dam. (See Appendix D-1 to D-5). The lower reaches of the watershed are sparsely to moderately developed while the upper reaches are less developed. The overall watershed was considered as rolling for our hydrologic computations.

b. <u>Discharge at Damsite</u> - Discharge at the damsite is over the spillways and through eight gates in the main dam embankment, two gates in the forebay dam and four gates in the wing dam.

1. Outlet Works (conduits):	Eight sluices in main dam embankment @ invert el. 275.3±
	One 60 inch dia. RCP in masonry forebay dam abut. @ invert el. 274.5
	One 42 inch sq. sluice in masonry forebay dam @ invert el. 277.8±
	Three 3.6'x6.0' sluices in wing dam @ invert el. 272
	One 4'x 6.0' sluice in wing dam @ invert el. 284
	Timber slide gate in powerhouse. Size & Invert Unknown
2. Maximum known flood @ damsite:	105,000 cfs. in Aug. 1955
3. Ungated spillway capacity @ top of dam el. 229.8: should be 299.8 (WJM)	55,000 cfs
4. Ungated spillway capacity @ test flood el.:	N/A
5. Gated spillway capacity @ normal pool el.:	N/A
6. Gated spillway capacity @ test flood el.:	N/A

7. Total spillway discharge @ test flood el.: N/A

8. Total project discharge @ test flood el. 304: 83,000 cfs.

c. Elevations (Feet Above Mean Sea Level)

1. Streambed @ centerline of dam: 268±

2. Maximum tailwater: 296.5

3. Upstream portal invert diversion tunnel: N/A

4. Recreation pool: 286 to 289

5. Full flood control pool: N/A

6. Spillway crest : 286.2

7. Design surcharge (original design): N/A

8. Top of dam (embankment): 299.8

9. Test flood design surcharge: 304

d. Reservoir

1. Length of maximum pool: N/A

2. Length of recreation pool: N/A

3. Length of flood control pool: N/A

e. Storage

1. Recreation pool: 350+ acre-ft.

2. Length of maximum pool: N/A

3. Spillway crest pool: 350 acre-ft.

4. Top of dam: 1400 acre-ft.

5. Test flood pool: 1960± acre-ft.

f. Reservoir Surface

1. Recreation pool: 55 acres (top of flashboards @ el. 289.2)

2. Flood control pool:	N/A
3. Spillway crest:	32 acres
4. Test flood pool:	140+ acres
5. Top of dam:	140 acres

g. Dam

1. Type:	Earthen embankment, masonry spillway with concrete wing dam and masonry forebay dam
2. Length (main dam):	660± ft.
3. Height (main dam):	32± ft. (max.)
4. Top width (embankment):	90± ft. (max.)
5. Side slopes:	Vertical upstream and downstream
6. Zoning:	N/A
7. Impervious Core:	N/A
8. Cutoff:	N/A
9. Grout curtain:	N/A
10. Other:	N/A

h. Diversion and Regulating Tunnel - N/A

i. Spillways

1. Type:	Broad crested granite or concrete weirs of trapezoidal cross-section
2. Length of weirs:	325 ft. (Main dam) 119 ft. (Forebay dam) 160 ft. (Wing dam)
3. Crest elevation:	286.2 (Main & Forebay Dams) 285.2 (Wing Dam)
4. Gates:	N/A

5. Upstream Channel: Riverbed

6. Downstream Channel: Riverbed with exposed ledge

7. General: Notch in main spillway 20 ft. long by 3 ft. deep (max.)

j. <u>Regulating Outlets</u> - Eight low level sluices

1. Invert: 275.3±

2. Size: 24.3 sq. ft. per opening - 3200 cfs approximate capacity @ test flood

3. Description: Irregular cross-section (See Appendix B-4)

4. Control Mechanism: Portable electric motor

5. Other: Gates in wing dam and forebay dam (See Section 1.3b)

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Available Data - The available data consists of drawings of the dam and highway bridge by the Connecticut Department of Transportation, Inventory Data by the State Water Resources Commission and a report entitled "Reconnaissance Engineering Geologic Investigation" by Robert L. Nelson of Foundation Sciences Inc., which was incorporated into the Canton Hydro-electric Project Feasibility Study by the Development and Resources Corporation. (This study may be viewed upon request to the Canton Conservation Commission, Canton Town Hall.)

b. Design Features - The drawings and reports indicate the design features noted in Section 1.

c. Design Data - There were no engineering values, assumptions, test results, or calculations available for the original dam construction or construction of the wing dam and powerhouse. Stability analyses were done for the dam in 1957 and, in 1978, for the hydroelectric feasibility study.

2.2 CONSTRUCTION

a. Available Data - The available data consists of as-built sketches of the dam by the Collins Company, and correspondence, a construction permit and a certificate of approval concerning the lowering of a portion of the main masonry spillway at the upstream end of the intake channel to the powerhouse. The 1957 plans for the highway bridge at the site also depict partial as-built conditions at the dam.

b. Construction Considerations - The lowering of a portion of the spillway as noted above was undertaken at the time of the highway bridge construction during the late 1950's. The right abutment of the bridge encroached somewhat upon the right end of the main dam spillway. The Collins Company became concerned that this would reduce flow to their powerhouse thereby reducing their power production capabilities. For this reason, a 20 foot portion of the masonry was removed to a maximum depth of three feet allowing for a greater flow to the powerhouse.

2.3 OPERATIONS

Flow in the Farmington River at Collinsville has been recorded daily by the U.S.G.S. since November, 1962. The flood of August, 1955 overtopped the earthen embankment (top of dam) by approximately 7 feet. The Collins Company kept formal operations records during the years the dam was used for power generation. However, in recent years (since 1966) no formal operations records are known to exist.

2.4 EVALUATION

a. Availability - Existing data was provided by the State of Connecticut Department of Transportation, the Water Resources Unit of the Connecticut Department of Environmental Protection and Mr. Dean C. Porterfield of the Canton Conservation Commission. The Perry Company may be in possession of further plans and/or data and correspondence left behind when the Collins Company vacated the factory buildings at the site, however officials of the company can not, at this time, find any relevant information among the voluminous materials in storage at the plant.

b. Adequacy - The final assessment of this dam is based on a review of existing data and on performance history, but primarily on the visual inspection, hydraulic computations of spillway capacity, and sound engineering judgement.

c. Validity - A comparison of the record data and visual observations reveals no observable significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The general condition of the dam is good. Some areas, obscured by overflowing conditions at the time of our inspection, require further investigation when river flows can be diverted through the gates and/or impounded upstream such that there is no water flowing over the spillways of the dams. Some other areas require minor maintenance. At the time of our inspection, there were approximately three inches of water flowing over the main spillway.

b. Dam:

Main Dam Masonry Spillway - The masonry section of the dam, at elevation 286.2, constitutes the main spillway, and is approximately 325 feet long, varying from between 6 and 18 feet high (Photo 1). Generally, it appears to be in good condition with no evidence of vertical or lateral displacement of the precisely cut granite gneiss blocks. No seepage through the masonry was observed, however, a clear observation of the downstream face and toe was prevented by water flowing over the crest.

Near its right end, the axis of the spillway is angled downstream slightly and continues for approximately 80 feet to its common abutment with the highway bridge. This 80 foot section serves as an inlet to the powerhouse intake channel. As previously mentioned, a 20 foot long notch, stepped to a maximum depth of three feet below the spillway crest exists in this section of the masonry spillway (Photo 7).

It was not possible to observe the upstream face of the spillway which was underwater, however, the available sections of it show the masonry to have a vertical upstream face with a heavy accumulation of silt against it. Also shown on the sections is the old timber dam immediately upstream of the present structure.

The spillway is capped by what appear to be granite blocks which are 11.3 feet across and which slope down to the upstream side on an approximate three horizontal to one vertical inclination (Photo 1). Near the downstream edge of the caps are drill holes used for the installation of posts for flashboards. The cap of the spillway appears to be in good condition with only a few places where pieces of the blocks have chipped away.

Earthen Embankment - To the left of the spillway is the earthen embankment section of the dam, which at elevation 299.8 is 13.6 feet above the spillway crest elevation. The embankment is separated into two portions by the forebay inlet, which is fed by eight sluice gates and spanned by a single span highway bridge. The upstream face of the embankment consists of a vertical masonry retaining wall with a concrete sluice gate bulkhead (Photo 3). The retaining wall has been partially reconstructed with concrete near the right end and along the top. The overall appearance and alignment of the wall is good, however one small birch tree is growing out of the wall about 20 feet to the right of the bulkhead. The condition of the wall where it abuts the concrete bridge structure, the concrete bulkhead and the natural ground at the left end of the dam is good. Two seeps were observed approximately one and three feet downstream of the juncture of the right end of the wall with the spillway (Photo 2).

The crest of the embankment is grass covered along its upstream half and paved along its downstream half. The crest appears to be in good condition with no signs of movement, settlement or cracking.

The downstream slope of the embankment is grass covered with a concrete retaining wall at its toe. The slope is in good condition with no signs of erosion or sloughing. The downstream face of the portion of the embankment housing the gates consists of a masonry retaining wall similar in construction to the upsteam retaining wall. This wall is in good condition, but has grass growing out of many of the joints in the masonry (Photo 4). Other concrete retaining walls along the right edge of the forebay, as shown on Sheet B-1, appear to be in good condition, with numerous weepholes which appear to be providing the proper hydrostatic relief to the dam embankment.

Concrete Wing Dam - The concrete wing dam extends downstream perpendicular to the major portion of the main spillway for 200 feet, where it adjoins the powerhouse (Photo 5). The 160 foot long broadcrested spillway is of trapezoidal cross-section and is at an elevation one foot lower than the main spillway. Flashboards may be accommodated along the length of the spillway crest. At the time of our inspection, only one approximately ten foot section of the flashboards at the right end of the spillway was intact, the rest having been broken away probably by ice in the winter and by spring flows (Photo 8). The abutment of the wing dam with the main spillway consists of a triangular shaped concrete section. The right abutment of the wing dam is 40 feet long and contains one mid level and three low level outlets. The concrete of the spillway portion is heavily spalled, while the right abutment is less so, except at the gate outlets where it is heavily spalled and some reinforcing is exposed. There is one extensive vertical crack at the joint between the spillway and the right abutment, however no seepage was observed at this crack.

Masonry Forebay Dam - The mill forebay is retained by a masonry dam on bedrock which extends from the highway abutment to the mill buildings (Photos 11, 13 and 14). Flashboards, broken in several places, top the dam. The dam appears in good condition with the exception of one possible leak at a 3/8 inch joint about 25 feet from the left abutment and about 2 feet from the crest (Photo 12). The water may, however, be water which passes over the crest, collects in a depression between the upper two courses of stonework and exits through a partially open joint.

c. Appurtenant Structures

Powerhouse - The presently non-functional powerhouse consists of a brick superstructure atop a concrete substructure, which appears to be founded on bedrock (Photos 5 & 6). Structurally, the powerhouse appears to be in good condition, with no significant cracking of the brick walls or concrete foundation and only minor spalling, however, it has been vandalized with only the easily accessable windows boarded up.

Discharge from the powerhouse is into a tailrace channel bounded on the left by a low concrete training wall (Photo 10) and on the right by a dry laid stone retaining wall and riverbank. The training wall is in poor condition, exhibiting cracking and undermining. Approximately 15 feet from the downstream end, a deteriorated portion of the wall allows water from the main channel to flow into the tailrace. The right retaining wall is in fair conditon with minor seeps about 15 and 25 feet downstream of the powerhouse. Erosion is occuring at a footpath where the stone retaining wall meets the concrete substructure of the powerhouse (Photo 9).

Gates - The eight low level gates for the main dam are operated by a portable electric motor kept on site. All appear to be operational. The two gates in the forebay dam are electrically operated by two seperate motors mounted on the platform, and they appear to be operational. The one mid level and three low level gates in the right abutment of the wing dam are manually operated and appear to be well maintained and operational, with the exception of the left low level gate, the valve stem of which is disconnected (Photo 6).

Route 179 Highway Bridge - A four span bridge extends from the right end of the embankment across to the right end of the main spillway, and a single span bridge spans the forebay inlet. The piers and abutments are founded on bedrock and the structures appear to be in very good condition.

d. Reservoir Area - At the right end of the dam, the shoreline is protected by concrete slabs and at the left end by dumped rock riprap. The shoreline of the impoundment basically consists of the natural riverbanks of the Farmington River which appear to be stable. It is possible that the reservoir storage may be somewhat reduced by sedimentation. There do not appear to be any significant potential upstream hazard areas.

e. Downstream Channel - The downstream channel consists of the broad boulder-strewn natural river channel with numerous bedrock exposures. There is heavy recreational usage of the river downstream of the dam.

3.2 EVALUATION

Based upon the visual inspection, it was possible to assess the dam as being generally in good condition, however, certain areas of the dam, such as the upstream and downstream faces of the main dam and concrete wing dam spillways, and the upstream face of the forebay dam, were obscured by overflowing conditions at the time of our inspection. The following features which could influence the future condition of the dam were identified.

1. Spalling and cracking of the concrete wing dam spillway and abutment is likely to continue and worsen in the future.

2. Through non-use and vandalism, the powerhouse area has fallen into a state of disrepair. It is easily accessible to the general public and presents a safety hazard to any trespassers.

3. The tailrace training wall is in a state of disrepair and there is significant erosion where the stone retaining wall to the right of the tailrace channel meets the concrete powerhouse substructure.

4. The valve stem to the left low level outlet in the wing dam abutment is disconnected, rendering it inoperable.

5. The birch tree growing from the upstream masonry retaining wall of the earthen embankment could eventually cause displacement of some of the masonry.

6. There is minor seepage through the main dam spillway near its abutment with the earthen embankment and possible minor seepage through the forebay dam spillway approximately 25 feet from its left abutment and two feet below the crest.

SECTION 4: OPERATIONAL PROCEDURES

4.1 REGULATING PROCEDURES

The eight sluice gates in the earthen embankment are normally at least partially open to supply water to the mill forebay. The Perry Company operates these gates as dictated by river flows by use of a portable electric motor stored on site. The Perry Company also maintains flashboards on, and operates the electrically powered gates in the forebay dam as needed to regulate the water level in the forebay. The local water ski club operates the gates in the wing dam, to lower the water level in order to install flashboards on the main dam and wing dam spillways. The flashboards usually break away during the winter and early spring and are reinstalled yearly during the late spring.

4.2 MAINTENANCE OF DAM

Although the State of Connecticut owns the dam, the Perry Company performs maintenance of the dam which basically entails cutting the grass on the embankment and removing debris from near the gates with a large grappling hook.

4.3 MAINTENANCE OF OPERATING FACILITIES

The gates in the embankment and in the forebay dam are maintained on an as-needed basis by the Perry Company. The water ski club regularly lubricates the manually operated gates in the wing dam as well as taking care of any other preventative or corrective maintenance of those facilities. In mid-July, 1979 the Perry Company was in the process of repairing the left low level gate in the forebay dam.

4.4 DESCRIPTION OF ANY FORMAL WARNING SYSTEM IN EFFECT

No formal warning system is in effect.

4.5 EVALUATION

The operation and maintenance procedures are generally satisfactory, however there are some areas requiring improvement. A formal program of operation and maintenance procedures should be implemented, including documentation to provide complete records for future reference. Also, a formal warning system should be developed and implemented within the time frame indicated in Section 7.1c. Remedial operation and maintenance recommendations are presented in Section 7.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. General - Collins Company Upper Dam is referred to as a run-of-river dam because the spillway spans the normal river channel and, during major floods, would be submerged by the tailwater.

b. Design Data - Water surface profiles for the river channel upstream and downstream of the Collins Company Upper Dam were obtained from 2 flood plain reports: 1) NED Army Corps of Engineers, "Flood Plain Information - West Branch and Farmington River, Canton, New Hartford, and Barkhamsted, Connecticut" dated May, 1977, and 2) H.U.D. - F.I.A. "Flood Insurance Study - Town of Canton, Connecticut," Proof Copy, dated February, 1979. The desired rating curves for flows up to the order of magnitude of the test flood (one-half PMF) were obtained utilizing the above water surface profiles as plotted on Appendix D-10.

c. Experience Data - The maximum flood at the site occurred during August, 1955, when a peak outflow of 105,000 cfs overtopped the dam about 7 feet, to elevation 307. At this time, the roadway bridge spanning the river at the dam was washed out, and subsequently replaced with the existing roadway bridge.

d. Visual Observations - No problem conditions were observed at the site which would affect the hydraulic performance of the facility.

e. Test Flood Analysis - The Collins Company Upper Dam watershed contains several lakes and reservoirs (See Section 1.3a) which could substantially reduce peak flows, especially when considering flows of a lesser magnitude than those due to a PMF storm. Considering the effect of these upstream reservoirs, it was determined that, while the reservoirs, with the exception of Colebrook, have very little reducing effect on peak inflows for a storm on the order of a PMF storm, there is considerable reduction of the peak inflow due to a one-half PMF storm (Appendix D-6). One-half PMF outflows for the upstream reservoirs were derived from Army Corps of Engineers Design Memorandums for the flood control dams and Phase I Inspection Reports for the other dams. Assuming simultaneous peaking of the various outflows, these were simply added together along with the contribution from the direct drainage area, yielding a conservative figure for peak inflow (D-6). The flashboards at the dam are designed to fail at a 2 to 3 foot head and, therefore, are not considered in the test flood analysis.

The test flood for this significant hazard, intermediate size dam is equivalent to one-half the Probable Maximum Flood (PMF). Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges", dated March, 1978, peak inflow to the reservoir is 83,000 cfs (Appendix D-6); peak outflow is 83,000 cfs with the dam overtopped 4 feet (Appendix D-12). Based upon our hydraulics computations, the spillway capacity is 55,000 cfs, which is approximately 66% of the routed test flood outflow. For this test flood, the spillway will operate under submerged conditions imposed by a tailwater stage to approximate elevation 296.5, which is approximately 10.5 feet above the spillway crest and approximately 3.3 feet below the top of the dam (D-12).

f. Dam Failure Analysis - Two conditions for dam failure were analyzed to determine the hazard classification: 1) Failure of the dam with the water level at the top of the dam, and 2) Failure of the dam with the water level at the spillway crest. The peak failure outflow of 60,000 cfs from the dam breaching with the water level at the top of the dam would result in a 1 foot rise in the water level at the impact area, i.e., from elevation 281 to elevation 282 (D-15). The 5 houses in the impact area have finished floor elevations at approximate elevation 280, which means that a flow in the river to elevation 281 before dam failure would be sufficient in itself to inundate the houses and cause evacuation of the residents in the impact area. Therefore, a breach of the dam causing a rise in the river level of 1 foot would cause no additional hazard in the downstream channel.

Utilizing the April 1978 "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", a failure of the dam with the water level at the spillway crest elevation would result in a peak failure outflow of 16,700 cfs and a corresponding rise of 6 feet in the water level from elevation 267 immediately before the breach to elevation 273 immediately after the breach. The 5 houses in the initial impact area are 7 feet above the level of the breach outflow, therefore the only hazard caused by a breach with the water level at the spillway crest elevation would be to persons downstream using the river for recreational purposes at the time of failure (D-15).

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations - Visual observations of the dam do not indicate any apparent stability problems. There is some significant deterioration of concrete on the wing dam and in other isolated areas, as described in Section 3.

b. Design and Construction Data - There is not enough design and construction data to permit an accurate in-depth analysis of the stability of this dam. A stability analysis performed by the Development and Resources Corporation (DRC) in their report entitled "Draft Final Report, Canton Hydroelectric Project, Feasibility Study" dated May, 1979, indicates that a "possible problem with regard to stability could exist" (Appendix B-32). The analysis indicates a factor of safety against overturning below "normally expected values". However, as the DRC did not have information on siltation, bedrock conditions, actual uplift pressures, and actual foundation configurations available when conducting the stability analysis, conclusions of possible stability problems may be inaccurate. The dam has withstood major floods of up to 7 feet above the top of dam elevation, therefore, it may be judged to be stable based upon the visual inspection and its past performance.

c. Operating Records - The operating records were not obtained.

d. Post Construction Changes - The original dam built in 1837 was raised 2 feet in 1849. In the late 1920's the powerhouse and wing dam were constructed, and the present Route 179 highway bridge was constructed in the late 1950's. The post construction changes probably did not decrease the stability of the dam.

e. Seismic Stability - The dam is in Seismic Zone 1 and according to the Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Condition - Based upon the visual inspection of the site and its past performance, the dam appears to be in good condition. No visual evidence of structural instability was observed in the masonry spillway sections or in the earth embankments, however, there is spalling of concrete on the wing dam. Other areas of concern include project discharge capacity and maintenance problems.

Based upon existing data and "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978, peak inflow to the impoundment is 83,000 cubic feet per second; peak outflow is 83,000 cubic feet per second with the dam overtopped 4 feet. Based upon our hydraulics computations, the spillway capacity is 55,000 cubic feet per second, which is equivalent to approximately 66% of the routed test flood outflow.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the dam must be based on existing data, visual inspection, past performance of the dam, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Sections 7.2 and 7.3 be implemented within two years of the owner's receipt of this report.

d. Need for Additional Information - There is a need for more information as recommended in Section 7.2

7.2 RECOMMENDATIONS

It is recommended that further studies be made by a registered professional engineer qualified in dam design and inspection pertaining to the following:

1. Examination of the downstream face and toe of the dam structures with the upstream pool elevation just below the spillway crest. Based upon his field observations, the engineer should then recommend any necessary repairs or renovations. Recommendations, made by the engineer, should be implemented by the owner.

2. Based upon the findings in 7.2.1 above, the engineer should determine if a stability analysis based upon detailed field determinations of actual uplift pressures and configurations of the dam foundation is necessary.

7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures - The following measures should be undertaken by the owner within the time frame indicated in Section 7.1.c, and continued on a regular basis.

1. Round-the-clock surveillance should be provided during and after periods of unusually heavy precipitation. A formal warning system with local officials for alerting downstream residents in case of an emergency should be developed.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference.

3. A program of inspection by a registered professional engineer qualified in dam inspection should be instituted on a biennial basis. The inspections should be comprehensive and include the operation of the low level outlet works.

4. Spalling and cracking of the wing dam spillway and abutment should be repaired.

5. The area of the powerhouse at the right end of the wing dam is in disrepair and should be effectively fenced off to prevent access by unauthorized personnel. The deterioration of the tailrace channel walls and erosion adjacent to the powerhouse right abutment should be repaired.

6. The valve stem to the left low level outlet of the wing dam should be repaired to render the outlet operable.

7. The birch tree growing from the upstream masonry retaining wall of the earthen embankment should be removed causing as little disturbance to the wall as possible.

7.4 ALTERNATIVES

This study has identified no practical alternatives to the above recommendations.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Collins Company Upper Dam

DATE: April 26, 1979

TIME: 9:00 AM

WEATHER: Overcast

W.S. ELEV._____ U.S._____ DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. Calvin Goldsmith	CG	Cahn Engineers, Inc.
2. Peter Heynen	PH	Cahn Engineers, Inc.
3. Theodore Stevens	TS	Cahn Engineers, Inc.
4. Gonzalo Castro	GC	Geotechnical Engineers, Inc.
5. Charles Osgood	CO	Geotechnical Engineers, Inc.
6.		

PROJECT FEATURE	INSPECTED BY	REMARKS
1. Main Dam Masonry Spillway	CG, PH, TS, GC, CO	
2. Earth Dam Section	CG, PH, TS, GC, CO	
3. Concrete Wing Dam	CG, PH, TS, GC, CO	
4. Mill Forebay Masonry Dam	CG, PH, TS, GC, CO	
5. Powerhouse Intake Channel	CG, PH, TS, GC, CO	
6. Powerhouse Tailrace Channel	CG, PH, TS, GC, CO	
7. Mill Forebay Intake Structure	CG, PH, TS, GC, CO	
8. Mill Forebay Outlet Structure	CG, PH, TS, GC, CO	
9.		
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

PROJECT Collins Company Upper Dam DATE April 26, 1979

PROJECT FEATURE Main Dam Masonry Spillway BY CG, TS, PH, GC, CO

AREA EVALUATED		CONDITION
~~DAM EMBANKMENT~~ MASONRY SPILLWAY		
Crest Elevation		286.2 MSL
Current Pool Elevation		286.4± MSL
Maximum Impoundment to Date (ELEV.)		304± MSL (AUG 1955)
Surface Cracks		SEVERAL CAPSTONES SLIGHTLY CHIPPED
Pavement Condition		N/A
Movement or Settlement of Crest		NONE OBSERVED
Lateral Movement		NONE OBSERVED
Vertical Alignment		APPEARED GOOD
Horizontal Alignment		APPEARED GOOD
Condition at Abutment and at Concrete Structures		GOOD - MINOR SEEPAGE AT LEFT ABUT.
Indications of Movement of Structural Items on Slopes		N/A
Trespassing on Slopes		N/A
Sloughing or Erosion of Slopes or Abutments		N/A
Rock Slope Protection-Riprap Failures		N/A
Unusual Movement or Cracking at or Near Toes		NONE OBSERVED
Unusual Embankment or Downstream Seepage		NOT OBSERVED - WATER SPILLING OVER CREST AT TIME OF INSPECTION
Piping or Boils		NONE OBSERVED
Foundation Drainage Features		NONE OBSERVED
Toe Drains		NONE OBSERVED
Instrumentation System		N/A

PERIODIC INSPECTION CHECK LIST

PROJECT COLLINS COMPANY UPPER DAM DATE APRIL 26, 1979

PROJECT FEATURE EARTH DAM SECTION BY CG, PH, TS, GC, CO

AREA EVALUATED		CONDITION
DAM EMBANKMENT		
Crest Elevation		299.8 MSL
Current Pool Elevation		286.4± MSL
Maximum Impoundment to Date (ELEV.)		304± MSL (AUG. 1955)
Surface Cracks		NONE OBSERVED
Pavement Condition		GOOD - ROADWAY
Movement or Settlement of Crest		NONE OBSERVED
Lateral Movement		NONE OBSERVED
Vertical Alignment		GOOD
Horizontal Alignment		GOOD
Condition at Abutment and at Concrete Structures		GOOD - MINOR SEEPAGE AT ABUT. WITH MASONRY OVERFLOW SECTION
Indications of Movement of Structural Items on Slopes		NONE
Trespassing on Slopes		NONE OBSERVED
Sloughing or Erosion of Slopes or Abutments		NONE OBSERVED
Rock Slope Protection-Riprap Failures		N/A
Unusual Movement or Cracking at or Near Toes		NONE OBSERVED
Unusual Embankment or Downstream Seepage		NONE OBSERVED
Piping or Boils		NONE OBSERVED
Foundation Drainage Features		NONE OBSERVED
Toe Drains		NONE OBSERVED
Instrumentation System		N/A

PERIODIC INSPECTION CHECK LIST

Page A-4

PROJECT COLLINS COMPANY UPPER DAM DATE APRIL 26, 1979

PROJECT FEATURE CONCRETE WING DAM BY CG, PH, TS, GC, CO

AREA EVALUATED		CONDITION
~~DAM EMBANKMENT~~ CONCRETE DAM		
Crest Elevation		285.2 MSL
Current Pool Elevation		286.4± MSL
Maximum Impoundment to Date (ELEV.)		304± MSL (AUG. 1955)
Surface Cracks		MINOR CRACKING OF CONCRETE
Pavement Condition (CONCRETE)		SIGNIFICANTLY SPALLED
Movement or Settlement of Crest		NONE OBSERVED
Lateral Movement		NONE OBSERVED
Vertical Alignment		GOOD
Horizontal Alignment		GOOD
Condition at Abutment and at Concrete Structures		LEFT ABUT. - COULD NOT OBSERVE RIGHT ABUT. - FAIR
Indications of Movement of Structural Items on Slopes		N/A
Trespassing on Slopes		AT RIGHT D/S ABUT OF POWERHOUSE PLATFORM
Sloughing or Erosion of Slopes or Abutments		EROSION AT RIGHT DOWNSTREAM ABUT. OF POWERHOUSE PLATFORM
Rock Slope Protection-Riprap Failures		N/A
Unusual Movement or Cracking at or Near Toes		NONE OBSERVED
Unusual Embankment or Downstream Seepage		NONE OBSERVED
Piping or Boils		NONE OBSERVED
Foundation Drainage Features		NONE OBSERVED
Toe Drains		NONE OBSERVED
Instrumentation System		N/A

PERIODIC INSPECTION CHECK LIST

Page A-5

PROJECT Collins Company Upper Dam DATE April 26, 1979

PROJECT FEATURE Mill Forebay Masonry Dam BY CG,PH,TS,GC,CO

AREA EVALUATED		CONDITION
DAM EMBANKMENT		
Crest Elevation		286±
Current Pool Elevation		286.4±
Maximum Impoundment to Date		OVERTOPPED AUG 1955
Surface Cracks		NONE OBSERVED
Pavement Condition		N/A
Movement or Settlement of Crest		NONE OBSERVED
Lateral Movement		NONE OBSERVED
Vertical Alignment		GOOD
Horizontal Alignment		GOOD
Condition at Abutment and at Concrete Structures		APPEARED GOOD
Indications of Movement of Structural Items on Slopes		N/A
Trespassing on Slopes		N/A
Sloughing or Erosion of Slopes or Abutments		N/A
Rock Slope Protection-Riprap Failures		N/A
Unusual Movement or Cracking at or Near Toes		NONE OBSERVED
Unusual Embankment or Downstream Seepage		ONE POSSIBLE SEEP APPROX. 25' FROM LEFT ABUT. AND 2' FROM CREST
Piping or Boils		NONE OBSERVED
Foundation Drainage Features		NONE OBSERVED
Toe Drains		NONE OBSERVED
Instrumentation System		N/A

PERIODIC INSPECTION CHECK LIST

PROJECT COLLINS COMPANY UPPER DAM DATE APRIL 26, 1979

PROJECT FEATURE POWERHOUSE INTAKE CHANNEL BY CG, PH, TS, GC, CO

AREA EVALUATED		CONDITION
OUTLET WORKS-INTAKE CHANNEL AND INTAKE STRUCTURE		
a) Approach Channel		FLOW TO POWERHOUSE IS THROUGH A 20' LONG NOTCH CUT INTO THE MAIN OVERFLOW SECTION AND OVER THE OVERFLOW SECTION TO THE POWERHOUSE FOREBAY
Slope Conditions		CONCRETE BLOCKS FOR MOST OF RIGHT SLOPE. EROSION WHERE BLOCKS ABSENT CHANNEL CONFINED ON LEFT BY WING DAM
Bottom Conditions		NOT OBSERVABLE
Rock Slides or Falls		NONE OBSERVED
Log Boom		NONE
Debris		COLLECTING NEAR GATES IN WING DAM
Condition of Concrete Lining		BRIDGE ABUTMENT (RIGHT SIDE) GOOD WING DAM (LEFT SIDE) SPALLED, DETERIORATED
Drains or Weep Holes		NONE OBSERVED
b) Intake Structure		
Condition of Concrete		SPALLED
Stop Logs and Slots		TRASH RACKS - SLIGHTLY CORRODED

PERIODIC INSPECTION CHECK LIST

PROJECT COLLINS COMPANY UPPER DAM DATE APRIL 26, 1979

PROJECT FEATURE POWERHOUSE TAILRACE CHANNEL BY CG, PH, TS, GC, CO

AREA EVALUATED		CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL		
General Condition of Concrete		FAIR - POWERHOUSE WALL POOR - TAILRACE TRAINING WALL
Rust or Staining		RUST AND STAINING OF POWERHOUSE WALL
Spalling		MINOR SPALLING OF POWERHOUSE WALL AND TAILRACE TRAINING WALL
Erosion or Cavitation		EROSION AND CAVITATION OF TAILRACE TRAINING WALL - UNDERMINING
Visible Reinforcing		NONE OBSERVED
Any Seepage or Efflorescence		NONE OBSERVED
Condition at Joints		O.K.
Drain Holes		TWO DRAIN HOLES IN POWERHOUSE WALL
Channel		TAILRACE CONFINED BY TRAINING WALL ON LEFT AND DRY-LAID MASONRY RETAINING WALL ON RIGHT
Loose Rock or Trees Overhanging Channel		FEW TREES - NO PROBLEM
Condition of Discharge Channel		APPEARED GOOD - TAILRACE DISCHARGES TO NATURAL RIVER CHANNEL

PERIODIC INSPECTION CHECK LIST

Page A-8

PROJECT Collins Company Upper Dam DATE April 26, 1979

PROJECT FEATURE Mill Forebay Intake Structure BY CG, PH, TS, GC, CO

AREA EVALUATED		CONDITION
OUTLET WORKS-INTAKE CHANNEL AND INTAKE STRUCTURE		APPROACH TO 8 SLUICE GATES IS NATURAL RIVER BOTTOM. FLOW FROM GATES BENEATH SINGLE SPAN BRIDGE TO FOREBAY.
a) Approach Channel		
Slope Conditions		N/A
Bottom Conditions		SAND, SILT, GRAVEL
Rock Slides or Falls		NONE
Log Boom		NONE
Debris		NONE OBSERVED
Condition of Concrete Lining		N/A
Drains or Weep Holes		N/A
b) Intake Structure		CONCRETE BULKHEAD ON U/S FACE OF EARTH DAM SECTION
Condition of Concrete		APPEARED GOOD
Stop Logs and Slots		8 SLUICE GATES OPERATED BY PORTABLE ELECTRIC MOTOR

PERIODIC INSPECTION CHECK LIST

Page A-9

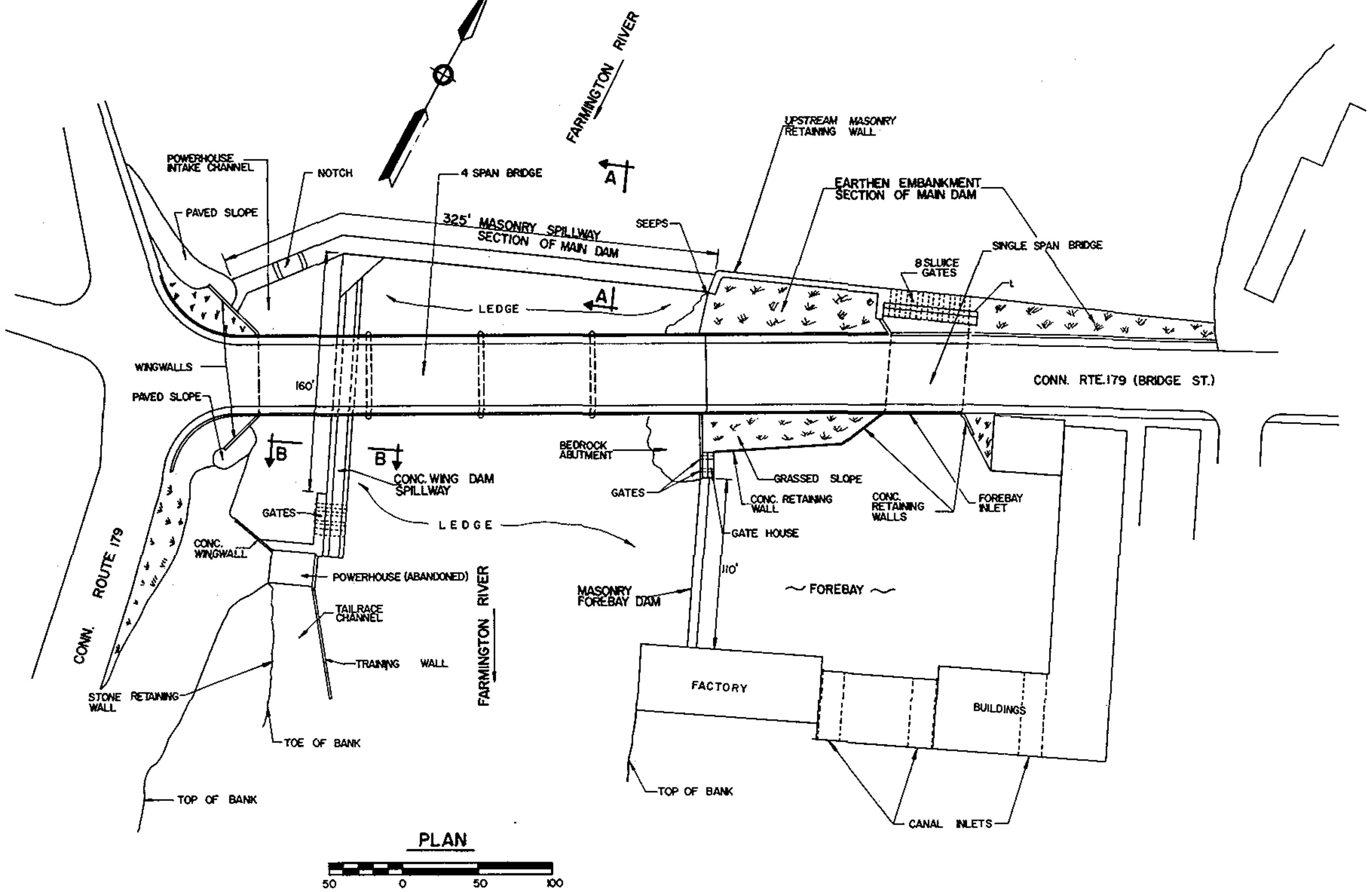
PROJECT COLLINS COMPANY UPPER DAM DATE APRIL 26, 1979

PROJECT FEATURE MILL FOREBAY OUTLET STRUCTURE BY CG, PH, TS, GC, CO

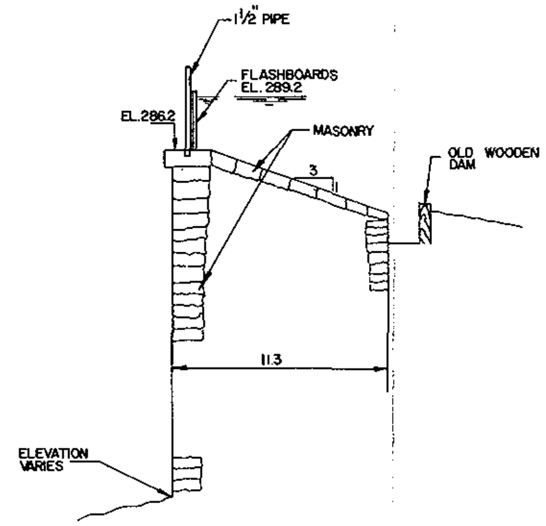
AREA EVALUATED		CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL		OUTLET FROM FOREBAY IS OVER MASONRY OVERFLOW SECTION, THROUGH A 42" SQUARE GATE AT RIGHT END OF MASONRY SECTION, THOUGH AN APPROX. 72" DIA. CONC. PIPE TO RIGHT OF MASONRY SECTION AND THROUGH 3 GATES INTO FACTORY BUILDINGS.
General Condition of Concrete		CONCRETE - GOOD CONDITION
Rust or Staining		NONE OBSERVED
Spalling		
Erosion or Cavitation		
Visible Reinforcing		
Any Seepage or Efflorescence		
Condition at Joints		CRACKING AT JOINT BETWEEN MASONRY AND CONC. AT RIGHT END OF FOREBAY DAM.
Drain Holes		
Channel		DISCHARGES INTO NATURAL ROCK RIVER BED
Loose Rock or Trees Overhanging Channel		NONE OBSERVED
Condition of Discharge Channel		GOOD

APPENDIX B

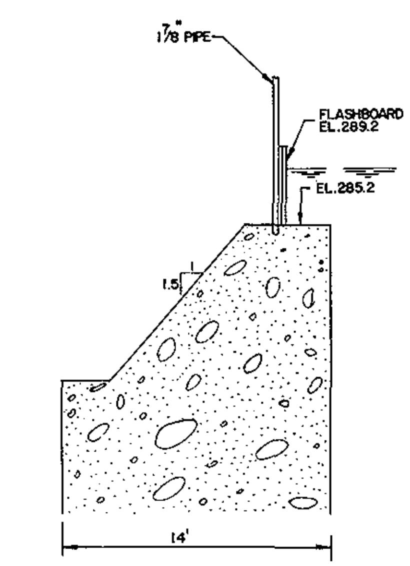
ENGINEERING DATA AND CORRESPONDENCE



PLAN



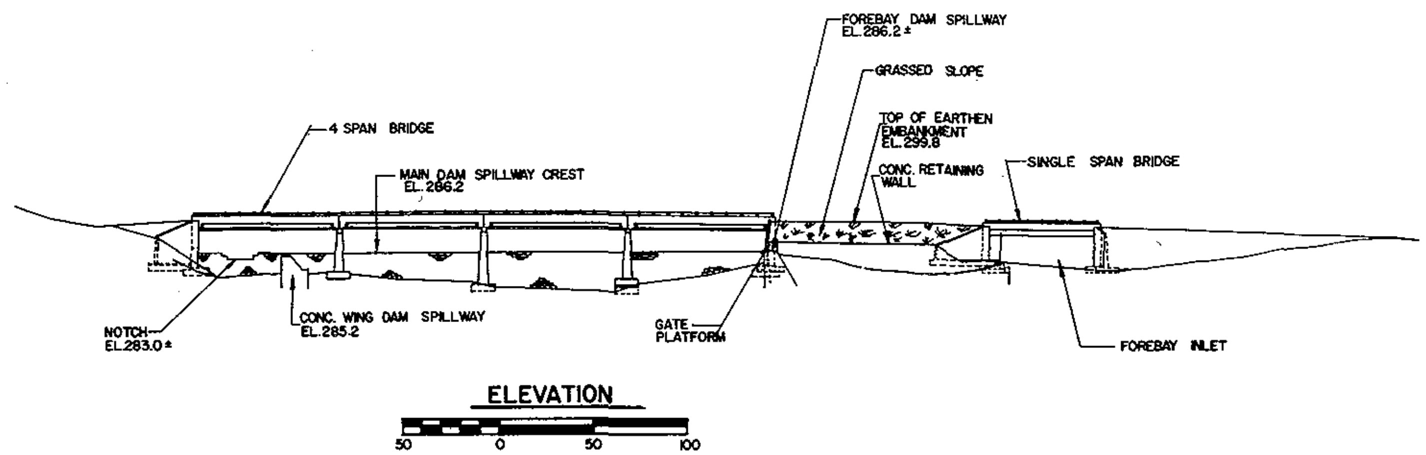
SECTION A-A
MAIN SPILLWAY
TYPICAL SECTION



SECTION B-B
CONC. WING DAM
TYPICAL SECTION

NOTES

1. THIS PLAN WAS COMPILED FROM EXISTING PLANS FOR THE DAM BY THE COLLINS CO. AND PLANS FOR THE ROUTE 179 HIGHWAY BRIDGE BY THE CONNECTICUT DEPARTMENT OF TRANSPORTATION DATED APRIL 30, 1979 AND JANUARY 11, 1974.
2. ELEVATIONS SHOWN ARE MEAN SEA LEVEL DATUM



ELEVATION

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.		
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS			
PLAN ELEVATION & SECTIONS			
COLLINS COMPANY UPPER DAM			
FARMINGTON RIVER	CANTON, CONNECTICUT		
DRAWN BY M. N	CHECKED BY CBG	APPROVED BY FmH	SCALE: AS NOTED DATE: JULY 1979 SHEET B-1

COLLINS COMPANY UPPER DAM

EXISTING PLANS

"Sketch of Upper Dam, Fore bay,
Powerhouse Canal and All Gates"
The Collins Company
Collinsville, Conn.
Oct. 14, 1936

"Flashboards - All Dams"
Cross Sections
The Collins Company
Collinsville, Conn.
June 9, 1942

"Power Plant - Westside River,
Sections - Canal Wall"
The Collins Company
Collinsville, Conn.
Dec. 19, 1956

"Road 765 over Farmington River"
Plan & Elevation
Connecticut State Highway Department
April 30, 1957

"Farmington River Bridge and Approaches"
Connecticut State Highway Department
1957

"Upper Dam"
The Collins Co.
Collinsville, Conn.
May 7, 1957

"Computing Sketch - Upper Dam"
The Collins Co.
Collinsville, Conn.
June 11, 1957

"Map Showing the Location of a Section of Highway to be
abondoned on Torrington Avenue (S.R.566)
Which Shall Revert to the Town"
Department of Transportation -Bureau of Highways
Jan. 11, 1974

SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
No Date	Files	Water Resources Commission Supervision of Dams	Inventory Data	B-3
June 7, 1930	Files	Collins Company	"Gate Opening in Bulkhead at Upper Dam."	B-4
June 11, 1957	Water Resources Commission	Newman E. Argraves State Highway Commissioner	Proposed alterations to weir	B-5
July 17, 1957	Collins Company	Water Resources Commission	Construction permit for alterations to weir	B-6
July 11, 1960	Collins Company	Water Resources Commission	Certificate of Approval regarding completed construction on alterations to weir	B-8
Dec. 26 1978	Clarence Korhonen Development and Resources Corporation	Robert L. Nelson Engineering Geologist Foundation Sciences, Inc.	"Reconnaissance Engineering Geologic Investigation - Canton Hydroelectric Project	B-9
May 3, 1979	Dean C. Porterfield Canton Conservation Commission	Mr. Clarence Korhonen Developement and Resources Corporation	Stability and Stress Analysis - Criteria and Summary (excerpt from <u>Canton Hydroelectric Project Feasibility Study)</u>	B-29

SUPERVISION OF DAMS
INVENTORY DATA

I-44

Inventoried By

Date

Name of Dam or Pond Collins Company DAM - UPPER

Code No.

Nearest Street Location RT 179

Town Canton

U.S.G.S. Quad.

LAT 41° 48.6

Name of Stream

LONG 72° 50.5

Owner State of Conn.

Address

Pond Used For Recreation - Water skiing

Dimensions of Pond: Width Length Area 50 Ac

Total Length of Dam 600 ft Length of Spillway 140 ft

Location of Spillway West side for main spillway gates on ...

Height of Pond Above Stream Bed 20 ft.

Height of Embankment Above Spillway 8 ft

Type of Spillway Construction Masonry & Concrete 3½ flashboards

Type of Dike Construction Road bed -

Downstream Conditions - Farmington River to Farmington

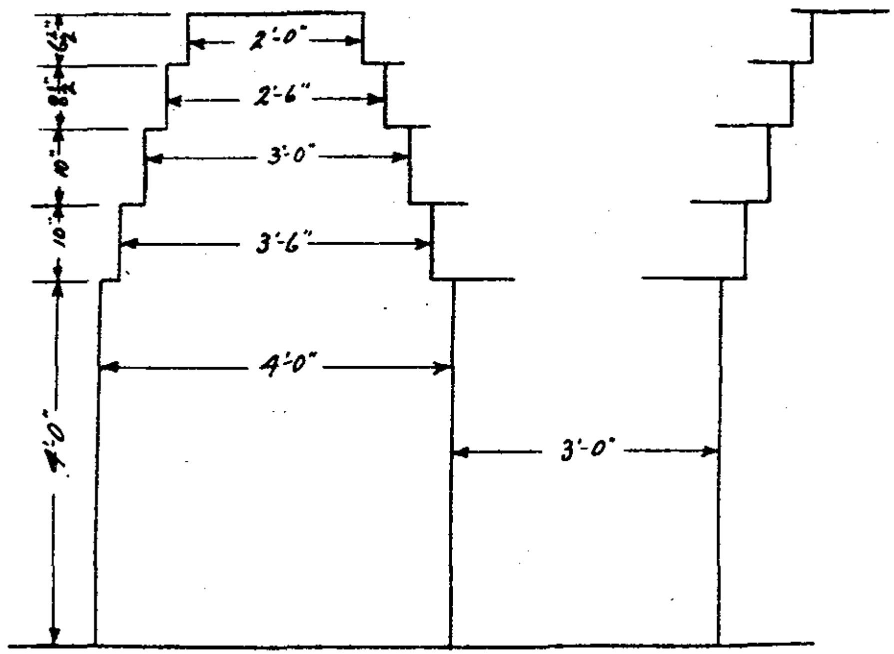
Summary of File Data Called Collins Company upper dam in file - Configuration changed slightly by

Remarks building new road

Struct Ht. 28 hydra Ht 26

normal Cap. 600 Max Cap 780

Would Failure Cause Damage? Yes Class B



Note:-
8 openings
24.27 Sq. Ft. per opening
194.16 " " Total "

CONTENTS:- GATE OPENING IN BULKHEAD AT UPPER DAM	The Collins Company Collinsville, Conn.	Date - 6-7-30 Scale - $\frac{1}{2}$" = 1'-0" Dr. by - J.C.M.	E1010

STATE OF CONNECTICUT

STATE HIGHWAY DEPARTMENT

STATE OFFICE BUILDING · HARTFORD 15, CONNECTICUT

June 11, 1957

Water Resources Commission
State Office Building
Hartford, Connecticut

Att: Mr. William S. Wise
Director

Re: Proposed Alterations to Weir at
Collins Company Dam on Farmington River
Town of Canton - Project 23-75

Gentlemen:

Submitted herewith are two copies of the general plan for a proposed bridge on Road 765 over the Farmington River in the Town of Canton.

The northwest abutment of the proposed bridge encroaches upon the submerged weir between the Collins Company pond and the forebay to their power plant on the west bank of the river. This weir is part of the original Collins Company dam at this location but has been entirely submerged by the water in the forebay of the power plant downstream.

Mr. Whitney, engineer for the Collins Company, has expressed the fear that the proposed reduction in the length of this submerged weir will have an adverse affect upon the quantity of water which can reach the turbine in their power plant. In order to compensate for the reduced capacity of the submerged weir, it is proposed to include the removal of the top of this weir for a distance of approximately 15' and a depth of approximately 3' as part of the construction of the new bridge.

Although this submerged weir no longer serves as part of the Collins Company dam, the plans are submitted for your consideration. If no permit is needed for the work outlined above, please advise the Highway Department. If you find that the work does come within your jurisdiction, it is requested that a permit be granted for the alterations.

Very truly yours,

NEWMAN E. ARGRAVES
State Highway Commissioner

By Robert A. Norton
Robert A. Norton
Hydraulics Engineer

Enc.
RAN:has

FORM D-5

STATE OF CONNECTICUT
WATER RESOURCES COMMISSION
Room 317 State Office Building
Hartford, Connecticut

CONSTRUCTION PERMIT FOR DAM

Date **July 17, 1957**

To: **Collins Company**

Collinsville,

Connecticut

Attention: Mr. Whitney

Dear Sir:

Your application for CONSTRUCTION PERMIT dated **June 11, 1957**, **submitted by the State Highway Department, together with their plan marked Sheet 3 of 25 Sheets, Project Number 23-75, covering proposed alterations to weir at Collins Company Dam on the Farmington River in the Town of Canton,**

copy of which is attached hereto, has been considered and the construction described therein is hereby approved under conditions which may be noted in the last paragraph of this permit.

This permit, with the attached application form and other enclosures, must be kept at the site of the work and made available to the Commission at any time during the construction. This permit covers the construction as described in the attached documents. If any changes are contemplated the Commission must be notified and supplementary approval obtained.

The Commission shall be notified [redacted] when the entire project is completed.

If the construction authorized by this construction permit is not started within two years of the date of this letter and completed within four years of the same date this permit must be renewed.

Your attention is directed to Section 5001 of the General Statutes: Obstructing Streams. No person shall, unless authorized by the superintendent, prevent the passing of fish in any stream or through the outlet or inlet of any pond or stream by means of any rack, screen, weir or other obstruction or fail, within ten days after service upon him of a copy of an order issued by the superintendent, to remove such obstruction. The address of the State Board of Fisheries and Game is 2 Wethersfield Avenue, Hartford 15, Connecticut.

The Commission cannot convey or waive any property right in any landsof the State, nor is this permit to be construed as giving any property rights in real estate or material or any exclusive privileges, nor does it authorize any injury to private property or the invasion of private rights or any infringement of federal, state or local laws or regulations.

Your attention is also directed to Section 23 of Public Act No. 364 of the 1957 Session of the General Assembly - Approval not to relieve owner from liability. Nothing in this chapter, and no order, approval or advice of the Commission or a member thereof, shall relieve any owner or operator of such a structure from his legal duties, obligations and liabilities resulting from such ownership or operation. No action for damages sustained through the partial or total failure of any structure or its maintenance shall be brought or maintained against the State, a member of the Commission or the Commission, or its employees, or agents, by reason of supervision of such structure exercised by the Commission under this chapter.

This permit is issued under the following special conditions.

The altered section of the weir will be properly capped.

WATER RESOURCES COMMISSION

By:______________________________
William S. Wise, Director

Note:
All correspondence relating to this project shall be in duplicate and addressed to the Commission.

cc: State Board of Fisheries and Game
State Highway Department

cc: Town Clerk of Canton

FORM D-7

STATE OF CONNECTICUT
WATER RESOURCES COMMISSION
Room 317, State Office Building
Hartford, Connecticut

CERTIFICATE OF APPROVAL

Date July 11, 1960

To: The Collins Company
Collinsville,
Connecticut

NAME OF STRUCTURE: Upper Collins Company Dam

This is to certify that the following construction work:
Alterations to weir at above dam, in accordance with plan marked sheet 3 of 25 Sheets, Project Number 23-75, and prepared by the State Highway Department,

on your property on the Farmington River

in the Town (s) of Canton

for which construction permit was issued July 17, 1960, has been completed to the satisfaction of this Commission and that such structure is approved as of date of this Certificate.

WATER RESOURCES COMMISSION

BY: ______________________
William S. Wise, Director

Note: The owner is required by law to record this Certificate in the land records of the town or towns in which the dam, dike or similar structure is located.
cc: State Highway Dept.

FOUNDATION SCIENCES, INC.

CABLE: FOUNSCIENCE
...OON

CASCADE BUILDING, PORTLAND, OREGON 97204
TEL. 503-224-4435

December 26, 1978

Development and Resources Corporation
455 Capitol Mall
Sacramento, CA 95814

Attention: Mr. Clarence Korhonen

Dear Mr. Korhonen

Enclosed for your use and distribution is one copy of each of our Final Reports entitled, "Reconnaissance Engineering Geologic Investigation, Phillips Hydroelectric Project, Croton Falls, New York" and "Reconnaissance Engineering Geologic Investigation, Canton Hydroelectric Project, Collinsville, Connecticut", dated December 26, 1978.

If you have any questions regarding our reports or require consultation, please do not hesitate to contact our office. We appreciate the opportunity to be of service to you on this project and the continued confidence you have in our services.

Very truly yours,

FOUNDATION SCIENCES, INC.

Robert L. Nelson

Robert L. Nelson
Certified Engineering Geologist (Oregon No. E502)

RLN:bh

Enclosures: 2 Final Reports
Quadrangle Report No. 16 (Canton Encl. No. 4)
Map (Canton Encl. No. 5)

	INITIAL	ACTION	INFO	FILE
	JJS			
	[illegible]			
	[illegible]			
	[illegible]		✓	
	[illegible]			
	[illegible]			
	[illegible]			
	[illegible]			
	RRB		✓	
A	T/M			

RECONNAISSANCE ENGINEERING GEOLOGIC
INVESTIGATION

CANTON HYDROELECTRIC PROJECT
COLLINSVILLE, CONNECTICUT

FOR

DEVELOPMENT AND RESOURCES CORPORATION
SACRAMENTO, CALIFORNIA

TABLE OF CONTENTS

LIMITATIONS

This reconnaissance evaluation of the foundation conditions as related to the present adequacy or deficiency of the dams and appurtenant works is based on conditions which are mostly underground and cannot actually be seen, nor were they tested.

There is some historical information available on the design and construction of the dams, but no information on the original site investigation or their operational performance. It must be understood, therefore, that the conclusions and recommendations presented are based in large part on indirect and incomplete information about the actual foundation conditions, even to a much larger degree than if an adequate subsurface investigation had been performed. The information in this study is not a certification or guarantee of the present suitability of the existing structures for their intended purposes or of the foundation conditions of proposed structures.

I. Regional Geology

The Canton Hydroelectric Project is located in the crystalline uplands of western Connecticut, part of an extensive area of structurally complex metamorphic and igneous rocks known collectively as the Appalachian Highlands. The crystalline uplands represent rocks of sedimentary origin, possibly silty shales, sandstones and carbonates which have been highly folded and faulted. The geologic history of the area from the (Cambrian) sedimentary origin is complex and involves at least one major period of crustal deformation and associated metamorphism and igneous intrusion which occurred during the Acadian Orogeny (Middle and Late Devonian). This mountain building produced the folds and gneiss domes which are characteristic of the area. The time from the end of the Acadian Orogeny to the Triassic Period was a period characterized by more or less gradual elevation of the rocks with erosion and deposition over the central and possibly western portions of Connecticut. These sedimentary rocks were then faulted and tilted eastward. A portion of these red Triassic sediments lie just east of the project site along the fault contact with the underlying metamorphic rocks. After this period of deformation in the late Triassic Period, continued erosion reduced the area to one of relatively low relief, caused development of major stream valleys like the Connecticut and exposed the complex crystalline rocks formed during the earlier geologic history. These rocks, some of which are exposed along the stream bed of the Farmington River at the site, consist of schists, gneisses and intrusives including granitic, pegmatitic and ultramatic rocks.

II. Site Geology

Geomorphology

The maximum relief at the site from the river bed to the adjacent hills is about 400 feet with hillsides sloping at approximately 25° to 30°. The height of the river bank in the lower right side of the reservoir area is about 15 feet. On the left side of the lower reservoir the river bank rises to the maximum elevation of the adjacent hills. Slopes around the upper reservoir immediately adjacent to the shore are relatively flat with 5 to 10 feet of relief adjacent to the flood plain areas. The river has a gradient of about 1.5° in the project area and has a rocky bed with numerous bedrock outcrops.

Lithology and Structure

Material at the site consists of bedrock, natural river bed alluvium, alluvium deposited as a result of the dams, rip rap (and other bank protection) and colluvium from the adjacent hillsides. These materials in relation to the existing facilities are shown on Figure 1.

The exposed bedrock consists of medium hard to hard, gray, medium grained garnite - muscovite - biotite - quartz - feldspar schist and gneiss with lenses of amphibolite and graphite - mica - quartz gneiss.

The rock hardness terminology used is :

> medium hard -- can be picked with moderate blows of the geology hammer.
> hard -- cannot be picked with geology hammer but can be chipped with moderate blows of the hammer.

The attitude of the bedrock foliation (bedding) and major joints was measured at three locations; just downstream from the sluice house at the lower dam, at the vicinity of the power house at the upper dam and at the highway cut on Rt. 179 just south of Collinsville.

Table 1 summarizes these measurements.

TABLE 1

Lower Dam Area

Bedding		Joints Set 1		Joints Set 2		Joints Set 3	
Strike	Dip	Strike	Dip	Strike	Dip	Strike	Dip
337°	64° SW					306°	75° NE
353°	66° SW						
345°	56° SW						

Upper Dam Area

Bedding		Joints Set 1		Joints Set 2		Joints Set 3	
Strike	Dip	Strike	Dip	Strike	Dip	Strike	Dip
020°	69° NW	020°	38° SE	327°	59° NE	308°	54° NE
024°	79° NW	013°	68° SE				
027°	60° NW						
000°	37° W						

Highway Cut

Bedding		Joints Set 1		Joints Set 2		Joints Set 3	
Strike	Dip	Strike	Dip	Strike	Dip	Strike	Dip
005°	67° NW	028°	48° SE	358°	24° NE		
015°	71° NW	055°	52° SE	340°	16° SW		

The information in Table 1 indicates that the attitude of the bedding displays a general north-south strike and a relatively steep westerly dip. This orientation is determined by the Collinsville Dome which is the main structural feature in the area. The table also indicates that there are possibly three predominant joint sets. It was not possible to determine, with the time available for study, which were the major and minor sets. In general, the joints are tight and spaced moderately close (1' - 3').

The natural river bed alluvium exposed along the banks consists of sandy gravel and rounded cobbles. In addition, there are accumulations of silty to clean fine sand deposited on the inside of bends in the river between the upper and lower dam and above the upper dam on the left side of the reservoir, north of the old railroad bridge. Also, there appears to be sandy gravel and cobbles at the water's edge around most of the upper reservoir. It is likely that the fine sandy alluvium was deposited as a result of the dam construction.

It was not possible to observe the material deposited directly upstream of the two dams but it likely consists of saturated, possibly loose fine sand. This material presumably extends to the original bottom elevation of the reservoir adjacent to the upstream face of the dams.

The rip rap and other bank protection placed around the reservoir consists of subangular to rounded cobbles and boulders, stone walls constructed of quarry rock and concrete walls. Bedrock is exposed along large segments of the river bank between the upper and lower dams, forming natural shoreline protection.

The colluvium, primarily exposed on the left shore of the reservoir upstream from the lower dam, consists of micaceous silty sand with scattered cobbles and boulders. Bedrock probably occurs at a shallow depth beneath the colluvium.

III. SEISMICITY

Because of their similar regional geology and earthquake history, the Phillips and Canton sites will be considered together in the following discussion of seismicity. The earthquake history of the area was reviewed using current information from the National Geophysical and Solar-Terrestrial Data Center of the National Oceanic and Atmospheric Administration and is summarized on Figure 2. Figure 2 shows the location of all earthquakes with an intensity of V or greater which have occurred from 1643 to 1978 within a 150 kilometer radius at each site. Based on this data, there have been a total of 44 seismic events in the last 335 years.

Table 2 summarizes this data relative to the total number and approximate frequency of occurrence of earthquakes of each intensity.

TABLE 2 -- Earthquake Frequency

Maximum Intensity *	V	VI	VII	VIII
Total number of Earthquakes	33	5	4	2
Approximate Frequency of Occurrence	10/50 yrs.	2/50 yrs.	1/50 yrs.	1/100 yrs.

*Modified Mercalli Intensity Scale of 1931.

To obtain design parameters for assessing the performance of existing or proposed structures under seismic loading, it is customary to discuss two hypothetic earthquakes, namely the maximum probable and maximum credible earthquake. Although the definitions of these two terms and the method of assigning a value to each are not consistent in practice, they are generally described as follows.

The maximum probable earthquake is the intensity at the site from the strongest earthquake that has ever occurred. This event is considered to have a reasonable possibility of occurrence during the design life of the structure and is based on the earthquake history and geology of the area. All structures should be designed to remain functional during such an earthquake, although minor repairs may be required.

The maximum credible earthquake is the strongest earthquake that can be expected to ever occur at the site based on understandable mechanisms, such as movement along a nearby large fault. Generally, the primary use of the maximum credible earthquake is to check the capability of the dam to retain water without catastrophic structural failure. The dam crest may be displaced significantly, and control structures may be rendered inoperable as long as they do not rupture and result in total failure of the dam. Repairs may be major.

The maximum probable earthquake is considered to be an intensity VIII event occurring at a distance of about 40 kilometers from the site. This was an actual earthquake which occurred SE of the Canton site (see Figure 2) although it is not possible to tell which fault may have caused the earthquake.

The maximum credible earthquake is considered to be an event occurring along a 25 kilometer straight line segment of a fault just south of the Phillips site within 10 kilometers of the dam. Although no historic earthquakes are known to have occurred along this fault, it is considered the most critical fault for the purpose of this study. A fault with at least the same straight line segment length occurs just east of the Canton site.

Table 3 summarizes the data used for these two earthquakes and presents related parameters.

The maximum probable earthquake developed in this summary as indicated in Table 3 produces a maximum bedrock acceleration at the site of .075 g. This acceleration is consistent with the seismic risk map of the Uniform Building Code which places the sites in Zone 1 (minor damage).

Because of the proximity of seismic risk Zones 2 and 3 to the project sites (see Seismic Risk Map, U.B.C.), the maximum credible earthquake with a resulting maximum bedrock acceleration of .2 g as developed in this summary is not considered overly conservative.

TABLE 3

Earthquake Design Parameters

	Fault Length	Fault Distance	Earthquake * Intensity	Earthquake * Intensity at Site	Maximum * Bedrock Acceleration at Site -g
Maximum Probable Earthquake	?	40 (Kilometers)	VIII	VI	.075
Maximum Credible Earthquake	25 (Kilometers)	10 (Kilometers)	IX	IX	.20

*Earthquake intensities, bedrock accelerations and attenuations based on data developed by Seed, Idriss and Kiefer, <u>Characteristics of Rock Motion During Earthquakes</u>, 1969.

IV. FOUNDATION CONDITIONS

Observations

Upper Power House -- There appears to be no cracking of the brick walls or concrete foundation. The concrete foundation and training walls for the power house are in contact with bedrock on the downstream side of the structure. Bedrock outcrops also occur immediately upstream from the power house. The left training wall on the river side is in contact with bedrock. Some cracks are visible on the inside of the left training wall. Leaks occur at the contact of the training wall and bedrock and in the stone wall which serves as the right training wall. Overflow water from theforebay strikes the adjacent bridge pier with high velocity. The main forebay walls just upstream from the power house are constructed directly on bedrock. The rest of the forebay walls were submerged and their condition or construction could not be observed.

Lower Power House and Gate House -- There appears to be no cracking of the brick walls, concrete foundation or concrete outlet works. No bedrock is actually visible in direct contact with concrete foundations of these two structures, however.

Power Canal -- Minor irregular cracks and deterioration occur on the right wall of the power canal every 10-15 feet ±. Cracking and one inch ± of vertical separation of a joint occurs about 200' downstream from the power house where a slight bend in the wall was constructed. Most of the left side of the power canal is a quarry-rock wall (no motar).

Sluice House -- There appears to be no cracking of the concrete foundation. The concrete foundation, in direct contact with bedrock, is visible on the downstream wall. There are bedrock outcrops both up and downstream from the sluice house. Leaks occur between the bedrock and concrete foundation on the downstream wall. The bedrock cliff downstream from the sluice house is very damp. A concrete retaining wall extends upstream from the sluice house for a considerable distance. It shows no bulging or settlement near the sluice house. Above the wall, sloping up to the abandoned railroad bed, rocks and boulder rubble are exposed.

Lower Dam -- The crest appears straight (no bulging in downstream direction) and level (no sags when viewed from upstream). It was not possible to examine the contact of the dam structure with the gate house or sluice house wall because of flowing water.

The even flow of water over the dam crest is disturbed by horizontal jets or sprays of water coming from the face of the dam. The sprays of water appear to be concentrated on the lower 1/3 of the dam face and arranged in continuous, somewhat irregular horizontal lines. No actual inspection at the concrete motar composing the dam could be made because of flowing water.

<u>Upper Dam</u> -- No bulging of the dam or settlement of the dam crest is apparent. No leakage appears to occur from between the stone blocks of the structure, however, water flowing over the crest prevented a more accurate determination. Bedrock is visible in direct contact with the stone blocks at each abutment and along most of the downstream toe of the dam. Some water was flowing from between the stone blocks and bedrock at the left abutment. Directly upstream from the right dam abutment for about 100 feet there is a sloping concrete slab which ajoins the highway bridge abutment. The shoreline upstream from the left dam abutment has rip rap for a considerable distance.

<u>Bedrock</u> -- Bedrock is exposed, in general, over the whole area downstream of the upper dam and in the proposed fish ladder location. Bedrock is not observed directly upstream of the dams except at the right abutment of the lower dam. Where bedrock is not exposed at the riverbed, it is expected to occur from 5 to 15 feet below the surface.

All of the schist and gneiss bedrock outcrops appear very hard and durable throughout the project area.

The strike of the bedding is oriented generally up and downstream or roughly perpendicular to the dam axses. The dip of the bedding is generally steep in a westerly direction. The strike of the joints is also generally perpendicular to the dam axses with the dip of the joint planes in a general upstream direction. The strike of the bedding and joints are generally parallel to portions of the forebay and canal walls which are oriented in a north-south direction. Joint and foliation planes intersect moderately frequently.

<u>Reservoir Areas</u> -- There was no evidence of slope movement or the potential for landsliding within the reservoir areas either between the upper and lower dams or upstream from the upper dam.

Old Railroad Bed -- From the lower dam to approximately 1500' upstream, the railroad bed appears to be constructed of rock rubble excavated from the nearby highway cut or is constructed directly on or very close to bedrock. The slope above the old railroad bed appears to be composed of large angular rocks excavated from the highway cut. From this point, to the old railroad bridge, the railroad bed becomes a slightly elevated embankment of sand and gravel.

V. CONCLUSIONS

Foundation Material

The foundation material beneath all the structures (dams, power houses, sluice house, forebays, power canals and etc) generally appears to have been of sufficient strength to support the loads imposed by these structures and other forces up to the present time. This is based on the fact that no settlement is detected along the dam crests. Also, no cracking is observed on any of the buildings. Most of the cracks on the right power canal wall, and on the training walls and foundations at the base of the upper power house and lower sluice house are likely related to erosion by water, or deterioration along joints and seams between successive concrete pours, and not to inadequate foundations. This conclusion is further supported by the hard and durable appearance of the bedrock throughout the area. Also, the available construction drawings indicate that the lower dam, together with the gate, power and sluice houses are founded on bedrock.

Regarding the apparent settlement in the right power canal wall, it is considered unlikely that poor foundation material has been the cause.

Although there are no drawings showing the upper dam foundation, it is considered very likely that the dam and appurtenant structures are all founded on bedrock. Drawings of the highway bridge, just downstream from the dam, indicate that the bridge footings are founded on hard bedrock. Also as mentioned previously, bedrock outcrops are extensive in the area.

Horizontal Movement

The attitude of the foliation and joints appears to present no adverse orientation which would cause horizontal movement of the dam or adjacent facilities along bedrock discontinuities. However, local variations in the attitude of these discontinuities are likely to occur. The effect of such variation on the stability of the bedrock foundation is impossible to assess without more detailed subsurface information.

Leakage

Significant leakage through the lower dam may be indicated by what appears to be horizontal jets or sprays coming from the face of the dam. It is also possible that such an appearance could be caused by water flowing over the crest, striking a rough spot on the face and being deflected outward. Without close examination of these areas of apparent leakage it is not possible to determine if they are detrimental to the strength or stability of the dam. Other areas of leakage observed, appear to present no serious threat to the structures involved since the water is flowing out between non-erosive material. If water flowing through the dam was causing progressive erosion of the masonry concrete, serious structural problems, could, of course, result.

Uplift Pressures

Uplift pressures in excess of normal tailwater conditions could occur if there is a confined zone of seepage beneath the structures, either between the structure and the bedrock or through the bedrock foundation. It was not possible to observe the areas immediately downstream from the structures for indication of seepage. As a consequence, and without any peizometers to monitor, it is impossible to determine if uplift pressures exist. The near vertical orientation of many of the foliation and joint planes in the rock, however, may tend to drain sufficiently to prevent the buildup of excess hydrostatic pressure at the toe of the dam.

Potential Penstock Location on Railroad Bed

The abandoned railroad bed appears to be constructed of material which would provide an adequate penstock foundation (see previous description).

Slope Stability

There appears to be a very low potential for landsliding from seismic loading or other causes within the reservoir areas or at the dams and appurtenant structures.

Liquifaction

It is possible that the material deposited directly upstream of the dams could liquify during an earthquake. This would cause maximum lateral earth pressures to develop against the base of the dams from the liquified sand (together with the horizontal earthquake loading).

VI. RECOMMENDATIONS

Foundation

Before final assessment of the adequacy of the foundations, it is recommended to inspect those areas of the facilities which were either not visible or inaccessible at the time of this study. These areas include mainly the interior foundations of the power houses, gate house and sluice house, and the face of the dams, forebay walls and other areas which were covered by flowing water. (Possibly inspect during low flow.)

Leakage

If possible, before final assessment of the seepage or leakage conditions is made, the dams should be observed during periods when there is a full head but water is not flowing over the crest.

Excavation

Rock excavation techniques will be required in bedrock. It is very difficult to access the potential for damage to the existing structures from blasting without better knowledge of the particle velocity propagation characteristics of the site and integrity of nearby masonry concrete or stone block structures. Based on studies by Nicholls, Johnson and Duval ("Blasting Vibrations and Their Effects on Structures", Bureau of Mines Bulletin 656, 1971), a safe blasting limit based on a scaled distance* of 50 ft/lbs$^{\frac{1}{2}}$ may be used provided a particle velocity of 2.0 inches per second is not exceeded in the foundation soil and/or rock affected by the blasting.

Before any blasting is undertaken, however, it is recommended that samples of the concrete be obtained from nearby structures for evaluation of its condition and the extent of alkali-silica reaction which has taken place. In addition, the face of the stone block structures should be examined closely for evidence of horizontal movement at joints. Also, instrumented blasts should be conducted at the site to determine the particle velocity propagation characteristics. This is especially important if excavation for a fish ladder is required very close to existing structures (the dam structure and highway bridge, for example).

---

*Scaled distance is obtained by dividing the distance in feet by the square root of the charge weight per delay interval in pounds.

If excavation is made close to the base of existing foundations, great care must be exercised to avoid under-cutting foliation planes, joint planes or other rock defects which could cause failure of the over-lying material by slippage along the defect.

Because rock excavation near the base of the dam could create a high risk situation regarding structure stability, it is recommended to investigate fish ladder designs which do not require rock excavation. It is recommended, therefore, to perform an accurate topographic survey of the rock surface in the area involved. It may be possible then, to choose an alignment for the fish ladder which will provide the required entry elevation and location, and at the same time require no, or very limited rock excavation.

If rock excavation is necessary, it is recommended to orient the line drilling along the planes of foliation. The rock will split easier in this direction.

Stability Analyses

It is recommended to perform stability analyses of the dam structure under both the maximum probable and maximum credible seismic loading. These should include other extreme loading conditions such as: maximum hydrostatic head, water flowing over crest and lateral loading due to possible liquifaction of the sand which has accumulated against the upstream face of the dam.

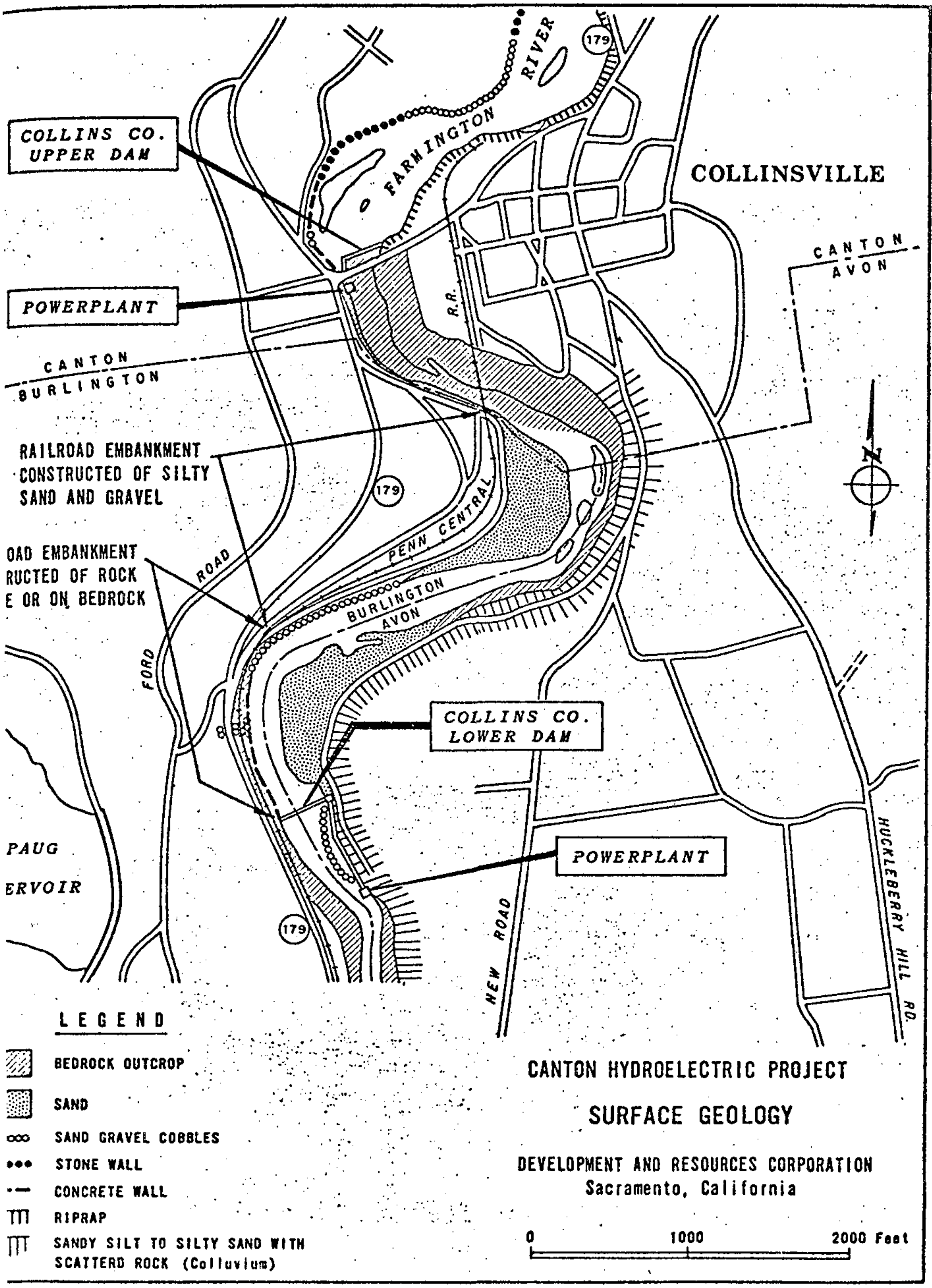
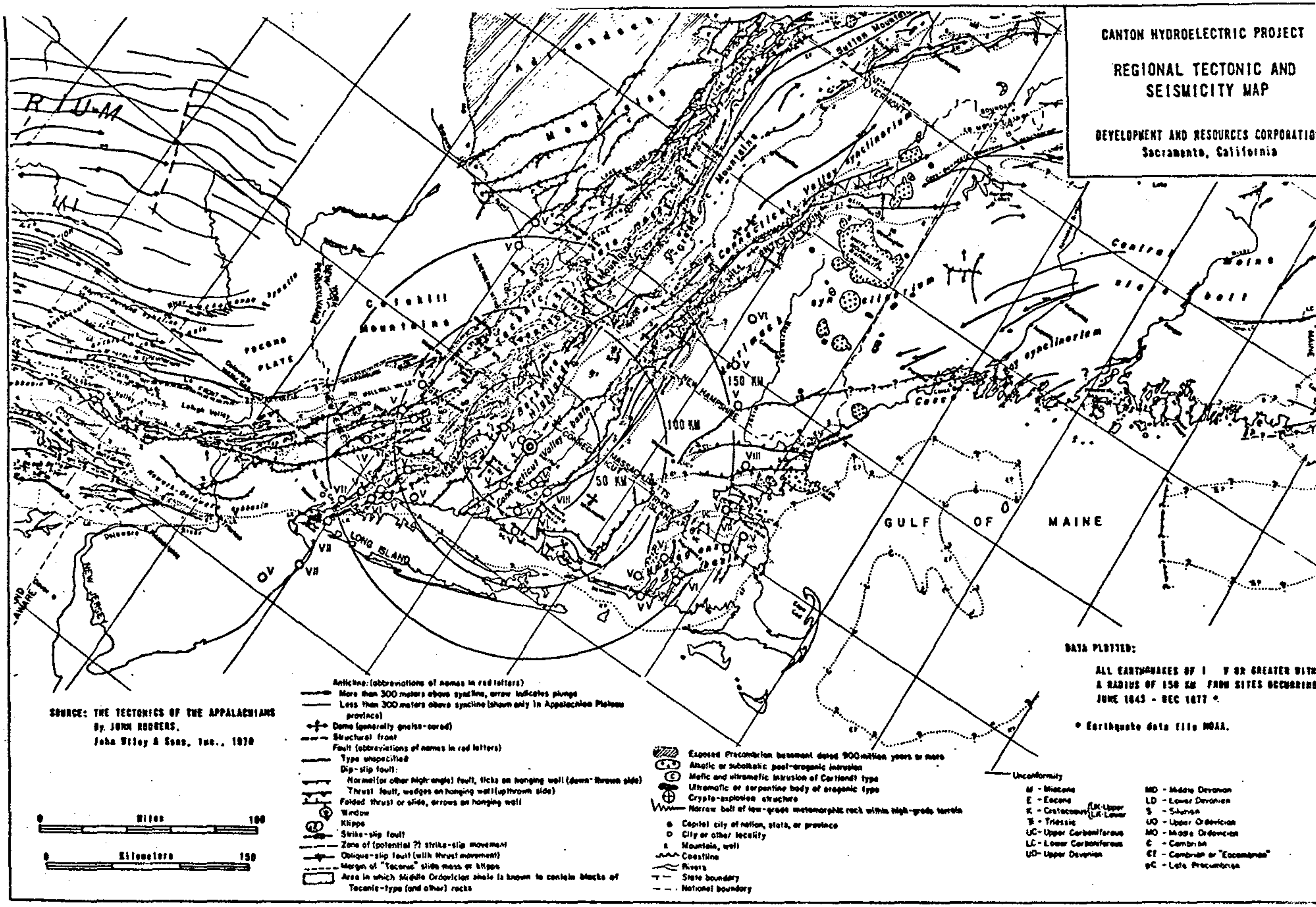


Figure App. B-1



CANTON HYDROELECTRIC PROJECT
REGIONAL TECTONIC AND SEISMICITY MAP
DEVELOPMENT AND RESOURCES CORPORATIO
Sacramento, California
GULF OF MAINE
LONG ISLAND
NEW HAMPSHIRE
MASSACHUSETTS
CONNECTICUT
VERMONT
NEW JERSEY
Catskill Mountains
Berkshire Highlands
Connecticut Valley Basin
150 KM
100 KM
50 KM
DATA PLOTTED:
ALL EARTHQUAKES OF I V OR GREATER WITHI
A RADIUS OF 150 KM FROM SITES OCCURRING
JUNE 1843 - DEC 1977 *.
* Earthquake data file NOAA.
SOURCE: THE TECTONICS OF THE APPALACHIANS
By. JOHN RODGERS,
John Wiley & Sons, Inc., 1970
Anticline: (abbreviations of names in red letters)
More than 300 meters above syncline, arrow indicates plunge
Less than 300 meters above syncline (shown only in Appalachian Plateau province)
Dome (generally gneiss-cored)
Structural front
Fault (abbreviations of names in red letters)
Type unspecified
Dip-slip fault:
Normal (or other high-angle) fault, ticks on hanging wall (down-thrown side)
Thrust fault, wedges on hanging wall (upthrown side)
Folded thrust or slide, arrows on hanging wall
Window
Klippe
Strike-slip fault
Zone of (potential ?) strike-slip movement
Oblique-slip fault (with thrust movement)
Margin of "Taconic" slide mass or klippe
Area in which Middle Ordovician shale is known to contain blocks of Taconic-type (and other) rocks
Exposed Precambrian basement dated 900 million years or more
Alkalic or subalkalic post-orogenic intrusion
Mafic and ultramafic intrusion of Cortlandt type
Ultramafic or serpentine body of orogenic type
Crypto-explosion structure
Narrow belt of low-grade metamorphic rock within high-grade terrain
Capital city of nation, state, or province
City or other locality
Mountain, well
Coastline
Rivers
State boundary
National boundary
Unconformity
M - Miocene
E - Eocene
K - Cretaceous {UK-Upper LK-Lower
Ŧ - Triassic
UC - Upper Carboniferous
LC - Lower Carboniferous
UD - Upper Devonian
MD - Middle Devonian
LD - Lower Devonian
S - Silurian
UO - Upper Ordovician
MO - Middle Ordovician
Є - Cambrian
ЄI - Cambrian or "Eocambrian"
pC - Late Precambrian
Miles 0 100
Kilometers 0 150

Figure App.

DIVERSION DAMS

Description and Condition

The Upper dam is approximately a maximum of 18 feet high and 350 feet long. This gravity overflow structure is composed of stone masonry with a vertical face on the downstream side. Steel pipes spaced at four feet have been installed at the crest of this structure to accommodate use of wooden flashboards up to 3.0 feet high. Visual inspection indicates that water passes through and between the wooden flashboards and, therefore, these units would need to be replaced for power generation. The dam itself, however, appears to be in good operating condition as no passage of water through the structure was noted and there have been no apparent lateral or vertical structure displacements. Plan drawings of the Collinsville Upper dam facility also indicate that the masonry structure is located directly in front of the original timber dam that was apparently left in place. No drawings or cross-sections of this older structure were available at the time of this study; and, it could not be visually inspected because of the river flows. The type and present condition of this timber structure could, therefore, not be assessed.

The Lower dam is a gravity overflow concrete structure approximately a maximum of 20 feet high with a crest length of 350 feet. During field reconnaissance, significant amounts of ravelling at the crest of this structure was indicated by the sharp jets and leakage of water passing over the crest. It should be further noted that the degree of deterioration at the crest is not known and that close examination of these areas would be recommended to determine the extent, if any, of leakage through the diversion structure. Progressive ravelling of the concrete caused by the passage of water through the structure could compromise the dam's structural integrity. No apparent vertical or horizontal structural displacements were noted during field inspections.

Dam Foundations

Visual inspection of the dam foundations at either the upper or lower sites could not be made because of flowing water. However, no lateral movement or settlement of the structures was noted during field reconnaissance trips. Field inspection further indicates that there are many rock outcroppings between the upper and lower dams. Based upon the geological report onthe area and visual observations, these rock formations are generally composed of schists and gneiss that are very hard and durable. Reference is made to the geology report included in Appendix B for a more complete description of the general regional and site geology.

An available detail drawing of the Lower dam indicates that this structure has been "keyed" into bedrock. These keys should prevent lateral displacement of the structure by the internal resistance of the key itself and the additional volume of foundation material that must be moved before the structure can slide. Furthermore, as judged by the strength of the surrounding rock formations, the structural capability of the foundation is considered to be competent and capable of withstanding the dam loadings and hydraulic flows to which it is subject.

The foundation for the Upper dam has been capable of sustaining the past dam and hydraulic loadings up to the present time. This is evidenced by the fact that no settlement or lateral movement of the dam could be noted during field reconnaissance trips. General surface geology report further indicates that there are many rock foundations in the vicinity of the Upper dam. Based on the Upper dam's past experience, coupled with the surface geology, it is felt that there is a strong possibility that the Upper dam is founded on firm hard bedrock which is capable of sustaining the required hydraulic and structure loads.

Stability Review

In order to assess the structural integrity of both diversion structures, analysis of each dam's structural loading conditions and stability were carried out. Calculations were based on the available section drawings and, for the purposes of calculation, each structure was considered to be homogeneous in nature. Table II-1 displays both the loading conditions and the design criteria utilized for determining each of the dam's factors of safety with regard to stability.

The loading cases displayed in these tables represent the maximum loads that each dam would be subject to under normal, seismic, and flood conditions. In order to assess earthquake loading conditions, seismic events of two different intensities have been used as a basis for review. Thus, Case II has been defined as a probable earthquake intensity while Case III defines the maximum credible seismic event. In order to account for vertical earthquake accelerations, both the weight of water above the structure and the dam itself was modified by an acceleration factor equivalent to 50% of the horizontal seismic loads applied. Case IV represents the peak river discharges based on the 50-year flood condition.

In all load cases silt is assumed to be in place and is taken into consideration in determining the resultant loads to apply. This is because it is considered probable that over the years significant amounts of silt and sand have accumulated against the upstream faces of the dams. Since it is not known how impervious the silt or foundation may be, full hydrostatic heads are used as a measure of the uplift forces. Thus, a straight line variation from headwater to tailwater is used in evaluating the magnitude of uplift forces. It should be noted, however, that if the silt material deposited on the upstream face of the dams is clay-like, it could be relatively imperious. This would, therefore, change the flow path of water beneath the structures, creating a differential in uplift pressure across the dam which would be something less than full hydrostatic. Since the actual differential in pressures is not known, both maximum and minimum possible uplift loads were utilized in the analysis of each diversion structure.

Based on the above loading conditions, factors of safety against overturning, uplift, actual sliding factors using stresses of each dam's base elevation were calculated. The results of these findings are displayed in Table II-2.

A possible problem with regard to stability could exist since calculations indicate that the dams' overturning factors of safety are below normally expected values. In view of these low factors, it is apparent that some type of anchorage at the toe of these structures most probably exists. The basis for this conclusion is also substantiated by the fact that both structures have withstood over 142 years and 65 years of flows respectively ranging to a maximum of at least 61,000 cubic feet per second (which occurred in the year 1955). This flow is approximately equivalent to a 250 year return frequency or a 0.4 percent chance of recurrence.

It is also possible that the bedrock which these structures are located on may tend to drain, thereby reducing the hydrostatic pressure and resulting uplift forces underneath the structures. It is recommended that the magnitude of pressures at the toe and heel of each structure be checked by field testing to determine the magnitude of actual uplift forces. Further review and structural analysis of each structure should then be carried out based upon the observed uplift pressures and actual anchorage conditions.

It is also necessary that a more detailed inspection of both Collinsville dams be made when the river flows can be diverted through the adjacent intake channels and/or sluice gates such that there is no water flowing over the crest of the dams. Such an inspection is required to verify that the downstream face of each structure is structurally intact and also to verify that there has been no undercutting at the downstream face at the interface with the bedrock. Signs of seepage should be looked for along with signs of deterioration of the cement mortar.
These activities would be included in the final site investigation and design stages of project implementation.

TABLE II-1

COLLINSVILLE DAMS

Design and Loading Criteria for Stability and Stress Analysis

Item	Design Loading Case I	II	III	IV
Flashboards	Yes	Yes	Yes	No
Water Surface Elevation				
Upper	U/S=289.2 D/S=266.8	U/S=289.2 D/S=266.8	U/S=289.2 D/S=266.8	U/S=294.7 D/S=286.7
Lower	U/S=269.7 D/S=253.7	U/S=269.7 D/S=253.7	U/S=269.7 D/S=253.7	U/S=275.2 D/S=269.7
Reservoir Silting at Dam				
Upper	282.5=assumed existing level	282.5=assumed existing level	282.5=assumed existing level	282.5=assumed existing level
Lower	264.7=assumed existing level	264.7=assumed existing level	264.7=assumed existing level	264.7=assumed existing level
Uplift Pressure	100 percent	100 percent	100 percent	100 percent
Seismic				
Horizontal	0	0.075	0.20	0
Vertical	0	0.0375	0.10	0
Stability				
Sliding Factor	0.7	0.7	0.7	0.7
Water Pressure	62.4 pcf	62.4 pcf	62.4 pcf	62.4 pcf
Saturated Soil Pressure	86 pcf	86 pcf	86 pcf	86 pcf

COLLINSVILLE UPPER AND LOWER DAMS

STABILITY AND STRESS ANALYSIS SUMMARY

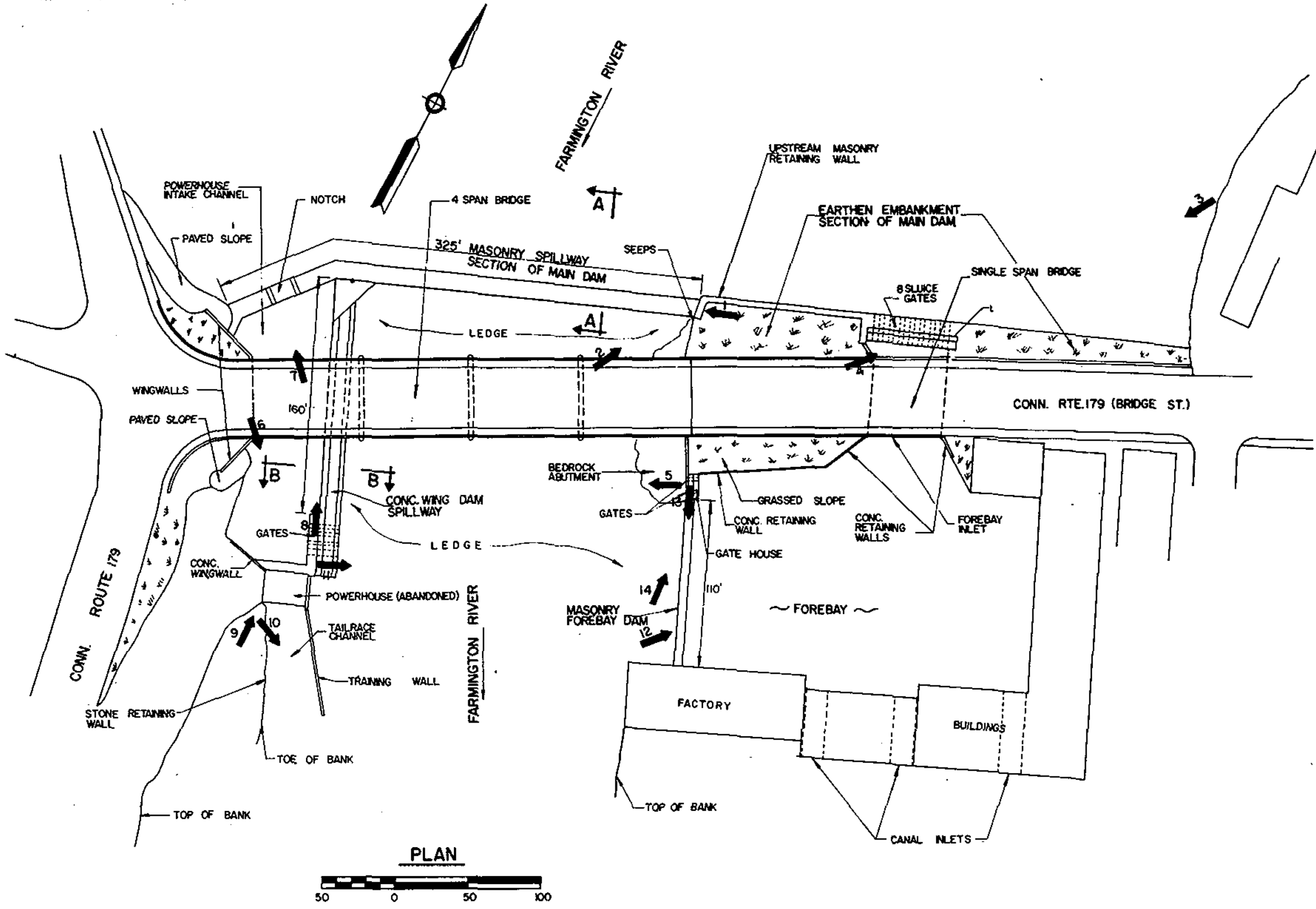
Item	Case Number			
	I	II	III	IV
LOWER DAM				
Stress (elevation 235.7)				
Heel (psi)	+24.8	+30.6	+40.2	+14.2
Toe (psi)	- 5.9	-13.2	-25.3	+ 7.4
Stability				
Uplift factor of safety	1.91	1.84	1.72	1.72
Overturning factor of safety with full uplift	1.21	1.06	.87	1.37
Overturning factor of safety without uplift	2.84	2.22	1.58	3.37
Sliding factor 2/	0	0	0	0
UPPER DAM				
Stress (elevation 267.83)				
Heel (psi)	+62.9	+69.9	+84.7	+44.5
Toe (psi)	-34.3	-42.7	-60.0	-25.6
Stability				
Uplift factor of safety	3.95	3.8	3.6	1.91
Overturning factor of safety with full uplift	.91	.76	.62	.93
Overturning factor of safety without uplift	1.32	1.04	.79	1.43
Sliding factor	.80	.99	1.36	.80
Actual sliding factor without uplift	.59	.73	.97	.38

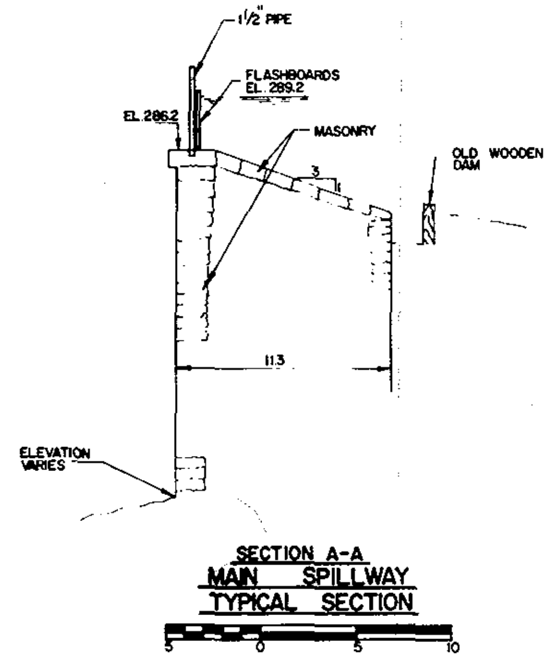
All stresses and safety factors with full hydrostatic uplift forces unless noted otherwise.

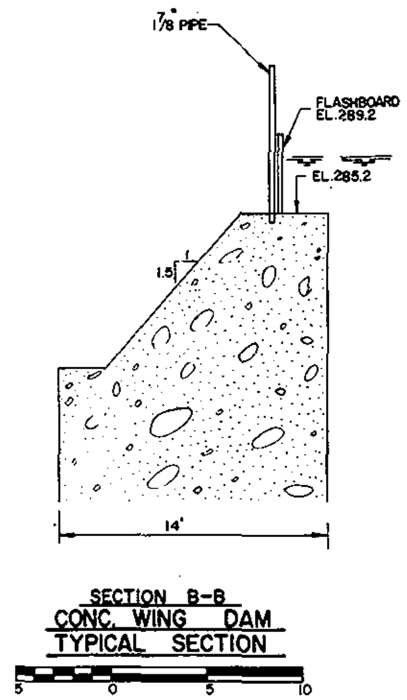
Lower dam keyed into bedrock which is assumed capable of resisting applied horizontal loads.

APPENDIX C

DETAIL PHOTOGRAPHS



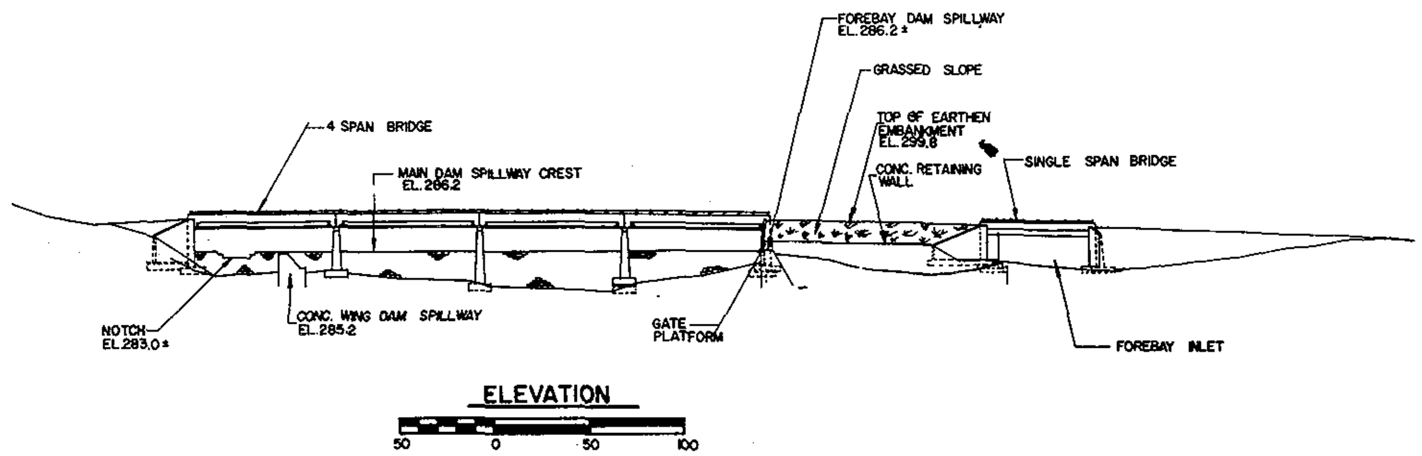




NOTES

1. THIS PLAN WAS COMPILED FROM EXISTING PLANS FOR THE DAM BY THE COLLINS CO. AND PLANS FOR THE ROUTE 179 HIGHWAY BRIDGE BY THE CONNECTICUT DEPARTMENT OF TRANSPORTATION DATED APRIL 30, 1979 AND JANUARY 11, 1974.
2. ELEVATIONS SHOWN ARE MEAN SEA LEVEL DATUM

3 ← 2 PICTURE NUMBER AND LOCATION



CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCATION PLAN OF PHOTOS

COLLINS COMPANY UPPER DAM

FARMINGTON RIVER — CANTON, CONNECTICUT

DRAWN BY	CHECKED BY	APPROVED BY	SCALE: AS NOTED
M. N	CRG	FMH	DATE: JULY 1979 — SHEET C-1

US ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
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CAHN ENGINEERS INC.
WALLINGFORD, CONN.
ENGINEER

NATIONAL PROGRAM OF
INSPECTION OF
NON-FED. DAMS

Collins Co. Upper Dam
Farmington River
Canton, Connecticut
CE# 27 595 KB
DATE July '79 PAGE C-1

PHOTO 1 - General view of masonry main dam spillway from right end of earthen embankment (April, 1979).

PHOTO 2 - Abutment of main dam spillway with embankment retaining wall. Note minor seeps in wall approximately one and three feet downstream of spillway crest (April, 1979).

PHOTO 3 - Upstream view of earthen embankment and eight sluice gates at left end of main dam (April, 1979).

PHOTO 4 - Downstream masonry headwall for sluice gates at inlet to forebay (April, 1979).

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CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 595 KB DATE July '79 PAGE C-2

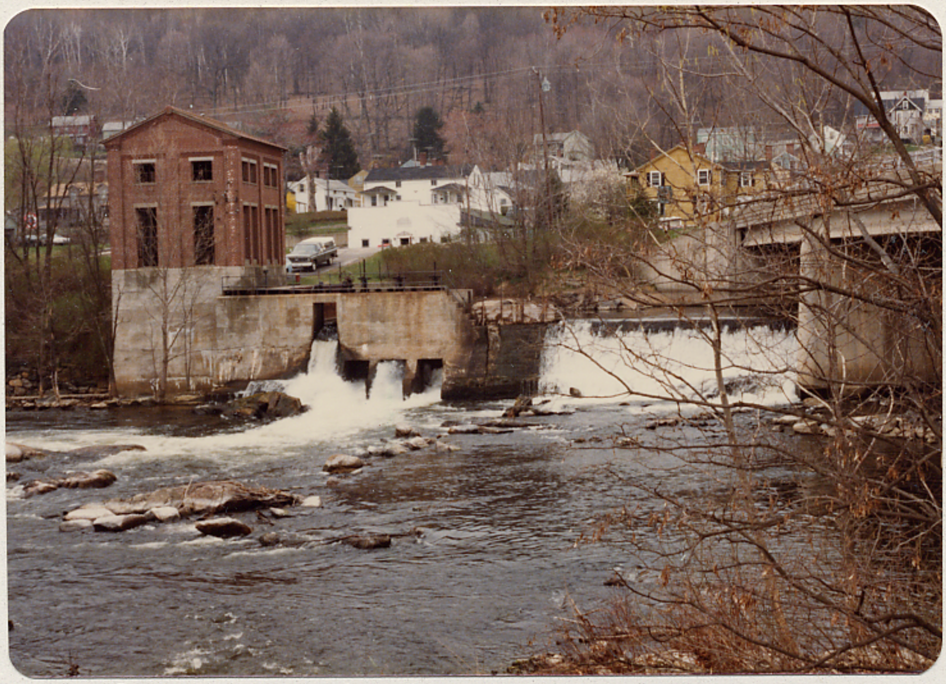

PHOTO 5 - Concrete wing dam and powerhouse at right end of dam. Note missing flashboards (April, 1979).

PHOTO 6 - Low level gate valves and trash racks at turbine intake. Note disconnected stem of left gate valve and corrosion of trash racks (April, 1979).

PHOTO 7 - Notch in main spillway at inlet to powerhouse intake channel contained by wing dam (April, 1979).

PHOTO 8 - Spillway crest of wing dam looking upstream from right wing dam abutment. Note broken flashboards (April, 1979).

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CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 595 KB DATE July '79 PAGE C-4

PHOTO 9 - Erosion at right powerhouse abutment (April, 1979).

PHOTO 10 - Concrete tailrace training wall. Note cracking and deterioration of concrete (April, 1979).

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CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 595 KB DATE July '79 PAGE C-5

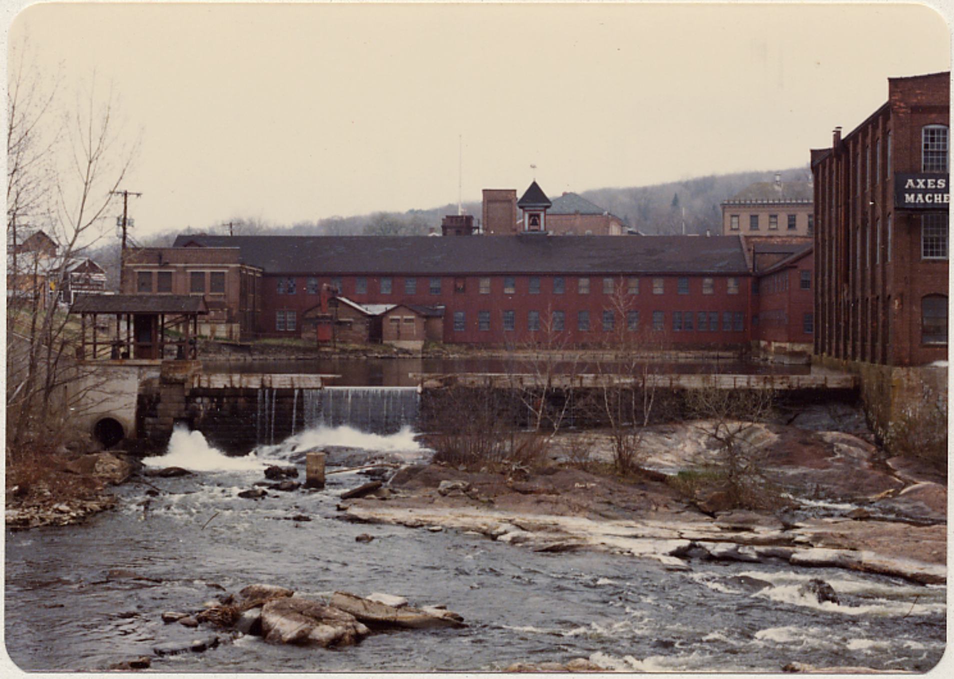

PHOTO 11 - General view of masonry dam for mill forebay. Note section of broken flashboards and low level outlets. Route 179 is at left (April, 1979).

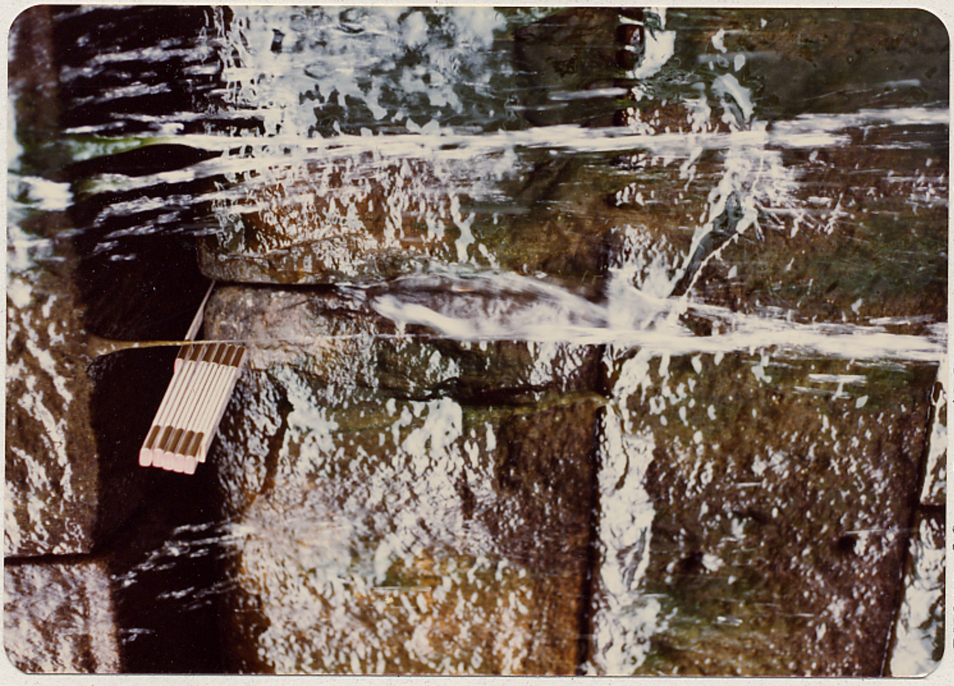

PHOTO 12 - Possible seep in forebay dam approximately 25 feet from left abutment and two feet below crest (April, 1979).

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PHOTO 13 - Forebay dam in winter. Note build-up of ice behind flashboards (January, 1979).

PHOTO 14 - Close-up of downstream face of forebay dam. Note broken flashboards, and natural ground downstream of right abutment and under Route 179 bridge (April, 1979).

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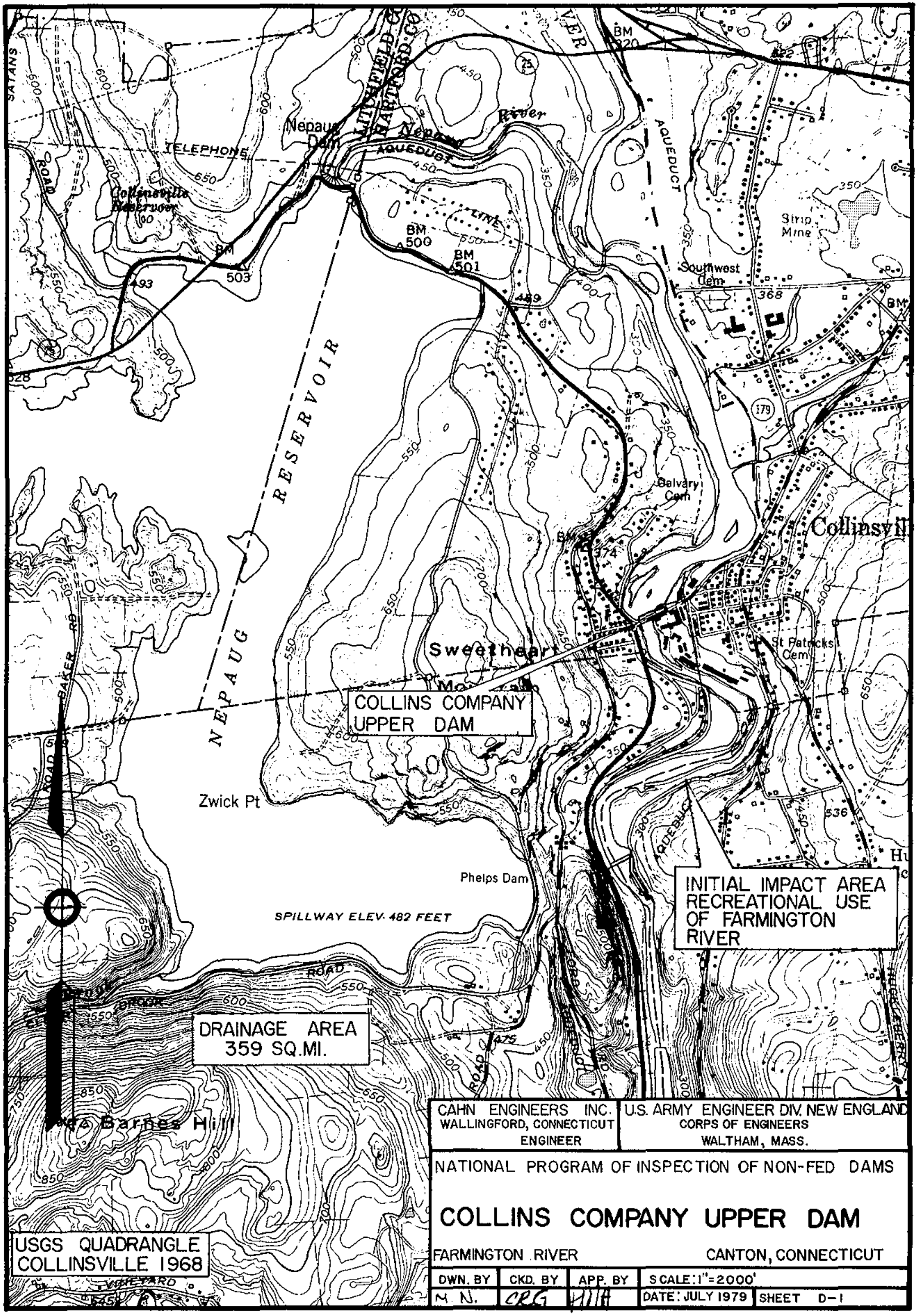
APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS



CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

COLLINS COMPANY UPPER DAM

FARMINGTON RIVER — CANTON, CONNECTICUT

DWN. BY	CKD. BY	APP. BY	SCALE: 1"=2000'	
M. N.	CRG	HJH	DATE: JULY 1979	SHEET D-1

HYDROLOGIC / HYDRAULIC INSPECTION

COLLINS CO. UPPER DAM, CANTON, CT.

2) PERFORMANCE AT TEST FLOOD CONDITIONS:

1) MAXIMUM PROBABLE FLOOD

a) WATERSHED CLASSIFIED AS "ROLLING".

NOTE: THIS CLASSIFICATION IS ASSIGNED TO THE FARMINGTON RIVER WATERSHED AT COLLINSVILLE, FOLLOWING A GRADUAL CHANGE IN ITS D.A. CHARACTERISTICS FROM "MOUNTAINOUS" AT COLEBROOK DAM TO "MOUNTAINOUS TO ROLLING" AT NEW HARTFORD TO "ROLLING" AT COLLINSVILLE.

b) WATERSHED AREA

THE COLLINS CO. UPPER DAM WATERSHED CONTAINS SEVERAL LAKES / RESERVOIRS WHICH MAY SUBSTANTIALLY REDUCE PEAK FLOWS SPECIALLY THOSE OF LESSER MAGNITUDE THAN THE PMF. THEREFORE, THE POSSIBLE EFFECT THAT THESE RESERVOIRS MAY HAVE ON THE PEAK FLOWS WILL BE CONSIDERED IN THIS ANALYSIS.

i) TOTAL D.A. $\approx$ 359 sq mi (U.S.G.S. HARTFORD OFFICE)

ii) D.A. OF WATERSHED REGULATED BY FLOOD CONTROL RESERVOIRS:

a.) COLEBROOK RES.: $(DA)_{CR} = 118$ sq mi (ACE DES. MEMO No 2, NOV. 1963)
b.) MAD RIVER RES.: $(DA)_{MR} = 18.2$ sq mi (ACE DES. MEMO No 1, OCT. 1960)
c.) SUCKER BROOK: $(DA)_{SB} = 3.43$ sq mi (ACE DES. MEMO No 1, JUN 1964)
(ii) TOTAL = 139.63

COLLINS CO. UPPER DAM

1,b-Cont'd) MAXIMUM PROBABLE FLOOD

iii) D.A. OF WATERSHED REGULATED BY OTHER RESERVOIRS:

a,) HIGHLAND LAKE (DIRECT D.A. D/S FROM SUCKER BROOK RES.) — $(DA)_{HL} = 3.54$ sq mi (ACE PH. I INSP.)
b,) SAVILLE (BARKHAMSTED) RESERV. — $(DA)_{SR} = 53.8$ sq mi (ACE PH. I INSP.)
c,) RICHARD'S CORNER (COMPENSATING) RES. (DIRECT D.A. D/S FROM SAVILLE RES.) — $(DA)_{RC} = 7.4$ sq mi (ACE PH I INSP.)
d,) NEPAUG RESERVOIR $(DA)_{NR} = 31.9$ sq mi (ACE PH. I INSP.)
(iii) TOTAL $= 96.64$ sq mi

iv) UNREGULATED (DIRECT) D.A. TO COLLINS CO. UPPER RES.:

$$D.A. = 359 - \underbrace{(139.6 + 96.6)}_{236.2} = 122.8 \approx \underline{123} \text{ sq mi}$$

c) FROM NED-ACE "PRELIMINARY GUIDANCE FOR ESTIMATING MAXIMUM PROBABLE DISCHARGES" - GUIDE CURVE FOR PMF - PEAK FLOW RATES:

i) PMF (CSM) FOR THE TOTAL D.A. : $(PMF)_{T} \approx 600$ cfs/sq mi

ii) PMF (CSM) FOR THE UNREG. D.A : $(PMF)_{R} \approx 1200$ cfs/sq mi

d) PEAK INFLOW

PEAK FLOW REDUCTION BY THE VARIOUS RESERVOIRS IS TAKEN WHEN AVAILABLE, OR OTHERWISE ESTIMATED BY APPROXIMATE FLOOD ROUTING, FROM THE ACE DESIGN MEMOS AND/OR PHASE I INSPECTION REPORTS FOR THE RESPECTIVE RESERVOIR. THE FLOOD CONTROL RESERVOIRS ARE ASSUMED EMPTY AT THE BEGINNING OF THE TEST FLOOD - (NED-ACE).
AN ACCOUNT OF THE OUTFLOW FROM THESE RESERVOIRS AND D.A. CONTRIBUTING TO

COLLINS CO. UPPER DAM

1,d - Cont'd) PEAK INFLOW

THE PEAK AT COLLINSVILLE (SITE OF THE COLLINS CO. UPPER AND LOWER DAMS), FOLLOWS:

i) COLEBROOK RESERVOIR:

FROM ACE "CONNECTICUT RIVER BASIN RESERVOIR REGULATION MASTER MANUAL" - APPENDIX J - FARMINGTON RIVER WATERSHED - JUNE 1970 AND/OR "COLEBROOK RIVER DAM & RESERVOIR", DESIGN MEMO. No. 2 - HYDROLOGY, NOV. 1963.

a,) INFLOW: PMF $=(Q_{P_1})_{CR} = 165000^{CFS}$ ½ PMF $=(Q_{P_1})^*_{CR} = 82500^{CFS}$

b,) OUTFLOW®: $(Q_{P_3})_{CR} = \underline{96000}^{CFS}$ $(Q_{P_3})^*_{CR} = \underline{16000}^{CFS}$

®(ASSUMED THE OUTFLOW FROM GOODWIN DAM - IMMEDIATELY D/S)

NOTE: THE COLEBROOK RESERVOIR OUTFLOWS FOR PMF AND ½ PMF (ASSUMED APPROX. EQUAL TO SPF) WERE FURNISHED FROM THE FIRST REFERENCE ABOVE BY NED-ACE, FOR THE FLOODS ROUTED ON THE CONDITION OF RESERVOIR EMPTY TO PERMANENT POOL AT THE BEGINNING OF THE FLOOD.

ii) MAD RIVER RESERVOIR:

FROM ACE "MAD RIVER DAM & RESERVOIR" - DESIGN MEMO No. 1 HYDROLOGY AND HYDRAULIC ANALYSIS - OCT. 1960

a,) INFLOW: PMF $=(Q_{P_1})_{MR} = 30000^{CFS}$ ½ PMF $=(Q_{P_1})^*_{MR} = 15000^{CFS}$

($\cong$ SPF = 15100 CFS)

b,) OUTFLOW: $(Q_{P_3})_{MR} \cong \underline{13000}^{CFS}$ $(Q_{P_3})^*_{MR} = \underline{430}^{CFS}$

NOTE: THE MAD RIVER RESERVOIR OUTFLOWS HAVE BEEN ESTIMATED

COLLINS CO. UPPER DAM

1,d - Cont'd) PEAK INFLOW

BY AN APPROXIMATE ROUTING OF THE FLOODS ON THE CONDITION OF RESERVOIR EMPTY TO PERMANENT POOL AT THE BEGINNING OF THE FLOOD. IT SHOULD BE NOTED THAT THE OUTFLOW FOR ½ PMF IS DISCHARGED TOTALLY THROUGH THE UNGATED CONDUIT OUTLET.

iii) SUCKER BROOK RESERVOIR AND HIGHLAND LAKE:

FROM ACE - HIGHLAND LAKE DAM, CT 00106 - PHASE I INSPECTION REPORT, DATED JUNE 1979 - WHICH IN TURN ASSUMED SUCKER BROOK RESERVOIR EMPTY AT THE BEGINNING OF THE TEST FLOOD.

a,) INFLOW TO SUCKER BROOK:

$PMF = (Q_{P_1})_{SB} = 6500^{CFS}$ $\quad$ $\frac{1}{2} PMF = (Q_{P_1})^*_{SB} \doteq 3250^{CFS}$

b,) INFLOW TO HIGHLAND (FROM SUCKER BROOK & DIRECT D.A.):

$PMF = (Q_{P_1})_{HL} \doteq 9500^{CFS}$ $\quad$ "$\frac{1}{2}$ PMF" $= (Q_{P_1})^*_{HL} \doteq 3600^{CFS}$

C,) OUTFLOW FROM HIGHLAND:

$(Q_{P_3})_{HL} \doteq \underline{6000}^{CFS}$ $\quad$ $(Q_{P_3})^*_{HL} \doteq \underline{2000}^{CFS}$

iv) SAVILLE (BARKHAMSTED) AND RICHARD'S CORNER (COMPENSATING) DAMS/RES.:

FROM ACE - SAVILLE DAM, CT 00376 AND RICHARD'S CORNER DAM, CT 00371 - PHASE I INSPECTION REPORTS, DATED SEPTEMBER, 1978, WHERE ROUTED OUTFLOWS FOR FULL PMF WERE DERIVED. APPROX. ROUTED OUTFLOWS FOR ½ PMF WERE ESTIMATED FROM DATA ON THESE REPORTS.

a,) INFLOW TO SAVILLE (BARKHAMSTED):

$PMF = (Q_{P_1})_{SB} = 78900^{CFS}$ $\quad$ $\frac{1}{2} PMF \doteq (Q_{P_1})^*_{SB} \doteq 39500^{CFS}$

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Computed By HLL   Checked By TJS
Field Book Ref. ______ Other Refs. CE#27-595-KB

Date 7/17/79
Revisions ______

COLLINS CO. UPPER DAM

1,d,iv - Cont'd) PEAK INFLOW - (SAVILLE AND RICHARD'S CORNER DAMS)

b,) INFLOW TO RICHARD'S CORNER (COMPENSATING), FROM SAVILLE AND DIRECT D.A.:

$PMF = (Q_{P_1})_{RC} \doteq 28200^{CFS}$ "½ PMF" $= (Q_{P_1})^*_{RC} \doteq 19800^{CFS}$

c,) OUTFLOW FROM RICHARD'S CORNER:

$^{\circledast}(Q_{P_3})_{RC} \doteq \underline{\underline{26400}}^{CFS}$ $(Q_{P_3})^*_{RC} \doteq \underline{\underline{17900}}^{CFS}$

⊛ NOTE: FROM C.E. APPROX. ROUTING. - REPORT GIVES $(Q_{P_3})_4 = 24360^{CFS}$

v) NEPAUG RESERVOIR

FROM ACE-NEPAUG DAM, CT 00370 AND PHELPS BROOK DAM, CT 00378 - PHASE I INSPECTION REPORT, DATED SEPTEMBER 1978, WHERE ROUTED OUTFLOW FROM THE NEPAUG RESERVOIR FOR FULL PMF WAS DERIVED. THE APPROXIMATED ROUTED OUTFLOW FOR ½ PMF WAS ESTIMATED ALSO FROM DATA ON THE PHASE I REPORT.

a,) INFLOW: $PMF = (Q_{P_1})_{NR} = 35300^{CFS}$ ½ PMF $= (Q_{P_1})^*_{NR} \doteq 17700^{CFS}$

b,) OUTFLOW: $(Q_{P_3})_{NR} = \underline{\underline{23000}}^{CFS}$ $(Q_{P_3})^*_{NR} \doteq \underline{\underline{10000}}^{CFS}$

vi) CONTRIBUTION TO PEAK FROM DIRECT D.A. TO COLLINS CO. UPPER DAM:

$PMF = (Q_{P_1})_{CC} \doteq 123 \times 600^{\circledast} = \underline{\underline{73800}}^{CFS}$

$½\ PMF = (Q_{P_1})^*_{CC} \doteq \underline{\underline{36900}}^{CFS}$

⊛ NOTE: 600 CFS/sqmi IS UNIT PMF FOR THE TOTAL D.A. (SEE p. D-2)

COLLINS CO. UPPER DAM

I,d-Cont'd) PEAK INFLOW

vii) PEAK INFLOW AT COLLINSVILLE (COLLINS CO. UPPER/LOWER DAMS)

a,) PMF $\simeq 359 \times 600 \simeq \underline{\underline{215000}}$ CFS

IT SHOULD BE NOTED THAT IN THE CASE OF FLOODS OF THE ORDER OF PMF, THE UPSTREAM LAKES/RESERVOIRS HAVE VERY LITTLE EFFECT IN REDUCING THE PEAK INFLOW. THIS IS EVIDENCED BY THE FLOW $Q \simeq 238000$ CFS (>PMF) OBTAINED BY ADDING THE PEAK OUTFLOWS OF THE U/S RESERVOIRS AND THE DIRECT D.A. FLOW. THE ASSUMPTION THAT ALL THE RESERVOIR OUTFLOWS PEAK SIMULTANEOUSLY IS SO CONSERVATIVE IN THIS CASE AS TO MAKE THE ERROR INVOLVED TO BE OF THE ORDER OF MAGNITUDE OF (AND EVEN UPSET) THE EXPECTED PEAK FLOW REDUCTION.

b,) "½ PMF" $\simeq \Sigma(Q_{P_3})^* + (Q_P)^*_{cc} \simeq \underline{\underline{83000}}$ CFS

THIS FIGURE, ALSO CONSERVATIVE BY ASSUMING SIMULTANEOUS PEAKING OF THE PARTIAL OUTFLOWS (VALLEY STORAGE IS NEGLIGEABLE), SHOWS A SUBSTANTIAL EFFECT IN THE PEAK REDUCTION BY THE RESERVOIRS (½ PMF $\simeq 359 \times 300 \simeq 108000$ CFS $\therefore \Delta Q \simeq 25000$ CFS)

A CHECK OF THE PEAK FLOW PRODUCED BY THE DIRECT D.A. TO COLLINS CO. DAMS SHOWS $Q_{PMF} \simeq 148000$ CFS, A PEAK OF LESSER MAGNITUDE THAN THE ABOVE.

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Computed By ___ Checked By TJS
Field Book Ref. ___ Other Refs. CE #27-595-KB

Date 7/18/79
Revisions ___

COLLINS CO. UPPER DAM

2) SPILLWAY DESIGN FLOOD (SDF)

a) CLASSIFICATION OF DAM ACCORDING TO NED-ACE RECOMMENDED GUIDELINES

i) SIZE: STORAGE (MAX.) $\cong 1400^{AC\text{-}FT}$ $(1000 < S < 50000^{AC\text{-}FT})$
HEIGHT $\cong 32'$ $(25 < H < 40^{FT})$

STORAGE: ESTIMATE BASED ON THE FARMINGTON RIVER PROFILE AND CROSS-SECTIONS AT RIVER MILES 40.92 AND 43.60, FROM THE ACE FLOOD PLAIN INFORMATION REPORT - "WEST BRANCH AND FARMINGTON RIVER - CANTON, NEW HARTFORD AND BARKHAMSTED, CONN." DATED MAY 1977 AND THE U.S.G.S. COLLINSVILLE, CONN. QUAD. SHEET, 1968.

(1) STOR. TO CREST OF SPILLWAY (ELEV. 286.2' MSL) BY AVE. X-SECT. AND LONG. RIVER PROFILE: $S \cong 350^{AC\text{-}FT}$

(2) SURCH. STOR. TO TOP OF DAM (ELEV. 299.8' MSL): SURF. AREA (USGS MAP) AT WL 289' MSL: $A \cong 55^{AC}$ (AT SPWY EL. ... $A \cong 32^{AC}$) AND AT CONTOUR EL. 300' MSL: $A \cong 140^{AC}$ ∴ AVE. AREA $\bar{A} \cong 86^{AC}$; SURCH. DEPTH $H \cong 14'$
∴ $\Delta S = 86 \times 14 \cong 1200^{AC\text{-}FT}$; ALSO, ASSUMING $\Delta S \cong$ VOL. OF TRUNCATED CONE: $\Delta S \cong 0.42 \times 140 \times 14 \cong 820^{AC\text{-}FT}$; ∴ TAKE $\Delta S \cong 1000^{AC\text{-}FT}$.

(3) ∴ $S_{MAX} \cong 350 + 1000 \cong 1350$ SAY, $S_{MAX} \cong \underline{1400}^{AC\text{-}FT}$

NOTE: U.S. INVENTORY OF DAMS, DATED 9/15/78, p.39, GIVES $S = 600^{AC\text{-}FT}$ (NORMAL) AND $S = 780^{AC\text{-}FT}$ (MAX.). THE FINAL DRAFT REPORT "CANTON HYDROELECTRIC POWER FEASIBILITY STUDY" BY DEVELOPMENT RESOURCES CORP., CALIF., GIVES STORAGE FOR DAM FAILURE ASSESSMENT OF $S \cong 200^{AC\text{-}FT}$.

HEIGHT: FROM ELEVS. ON COLLINS CO. DWG. NO. B4060 "UPPER DAM" - MAX. HEIGHT OF SPILLWAY SECTION $H_S = 18'$ (EL. 82.0' ≡ 268.2' MSL TO EL. 100' ≡ 286.2' MSL); FROM STATE HIGHWAY DEPT. PLANS/PROFILES OF ROAD 765 FOR CONSTRUCTION OF NEW FARMINGTON RIVER & COLLINS CO. INTAKE CANAL BRIDGES, TOP ELEV. OF DAM IS (±) EL. 299.8' MSL* OR, (±) 14' ABOVE THE SPILLWAY CREST. ∴ MAX. HEIGHT OF DAM: $H \cong 18 + 14 \cong \underline{32'}$

COLLINS CO. UPPER DAM

2.a - Cont'd) CLASSIFICATION

ii) HAZARD POTENTIAL: THE DAM IS LOCATED (±) 2000' U/S FROM FIVE NEW HOUSES (ONE STILL UNDER CONSTRUCTION) WHICH ARE (±) 13' ABOVE THE RIVER NORMAL W.S. (i.e., F.F. IS ASSUMED AT (±) 280' MSL). THE FARMINGTON RIVER HAS A HIGH RECREATIONAL USE.

iii) CLASSIFICATION:

SIZE: INTERMEDIATE

HAZARD: SIGNIFICANT

NOTE: ALTHOUGH THE D/S POTENTIAL IMPACT AREA INVOLVES THE ABOVE MENTIONED HOUSES, THE DAM IS A "RUN-OF-RIVER" DAM WHERE THE SPILLWAY LENGTH EQUALS APPROX. THE NORMAL STREAM WIDTH AND ON MAJOR FLOODS, IS SUBMERGED BY THE TAILWATER (AUG. 1955 FLOOD) AND, THEREFORE, THE EXPECTED FLOODING EFFECT UPON FAILURE OF THE DAM PROBABLY WILL NOT REPRESENT A HIGH HAZARD. LOSS OF LIFE D/S FROM THE DAM BECAUSE OF THE HIGH RECREATIONAL USE OF THE RIVER, IS CONSIDERED A SIGNIFICANT HAZARD. THIS CLASSIFICATION WILL BE MODIFIED IF THE ANALYSIS SO INDICATES.

b) SDF = "½ PMF" ≅ 83000 cfs   PMF ≅ 165000 cfs

3) SURCHARGE AT PEAK INFLOW

a) PEAK INFLOW: Q_P = 83000 cfs.

b) SPILLWAY (OUTFLOW) RATING CURVE:

ESTIMATES OF SURCHARGE/SPILLWAY RATING CURVE FOR THIS TYPE OF DAM, PARTICULARLY FOR FLOODS OF THE ORDER OF MAGNITUDE OF THE TEST FLOOD, REQUIRE DETERMINATION OF THE TAILWATER

COLLINS CO. UPPER DAM

3.b - (Cont'd) OUTFLOW RATING CURVE

WHICH SUBMERGES THE SPILLWAY (D/S CHANNEL RATING CURVE).

WATER SURFACE PROFILES FOR THE RIVER CHANNEL U/S AND D/S OF THE COLLINS CO. UPPER DAM THAT ALLOW DETERMINATION OF THE DESIRED RATING CURVES FOR FLOWS UP TO THE ORDER OF MAGNITUDE OF THE TEST FLOOD (Q_P) ARE AVAILABLE FROM TWO FLOOD PLAIN REPORTS: (1) NED-ACE, "FLOOD PLAIN INFORMATION - WEST BRANCH AND FARMINGTON RIVER - CANTON, NEW HARTFORD AND BARKHAMSTED, CONNECTICUT", DATED MAY 1977; AND (2) HUD-FIA "FLOOD INSURANCE STUDY - TOWN OF CANTON, CONNECTICUT," PROOF COPY, DATED FEB. 1979.

THE U/S AND D/S W.S. ELEVATIONS FROM THE AVAILABLE PROFILES ARE USED AS PLOTTING POINTS FOR THE COLLINS CO. UPPER DAM SPILLWAY (OUTFLOW) RATING CURVE SHOWN ON NEXT PAGE (D-10). A TABULATION OF THE SELECTED DATA IS AS FOLLOWS:

DISCHARGE (Q-CFS)	U/S W.S. ELEV. (MSL) ACE	U/S W.S. ELEV. (MSL) HUD	D/S W.S. ELEV. (MSL) ACE	D/S W.S. ELEV. (MSL) HUD
80000	305	303.5	296.2	296.2
41000	296.1	297.4	289	289
29000	—	295	—	286.8
13000	—	291.5	—	282
*105000	307 (±)	—	—	—

*HIGH WATER MARK - FLOOD OF AUGUST 1955 (UNMODIFIED)

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Project NON-FEDERAL DAMS INSPECTION Sheet D-10 of 15

Computed By HAL Checked By TJS Date 7/19/79

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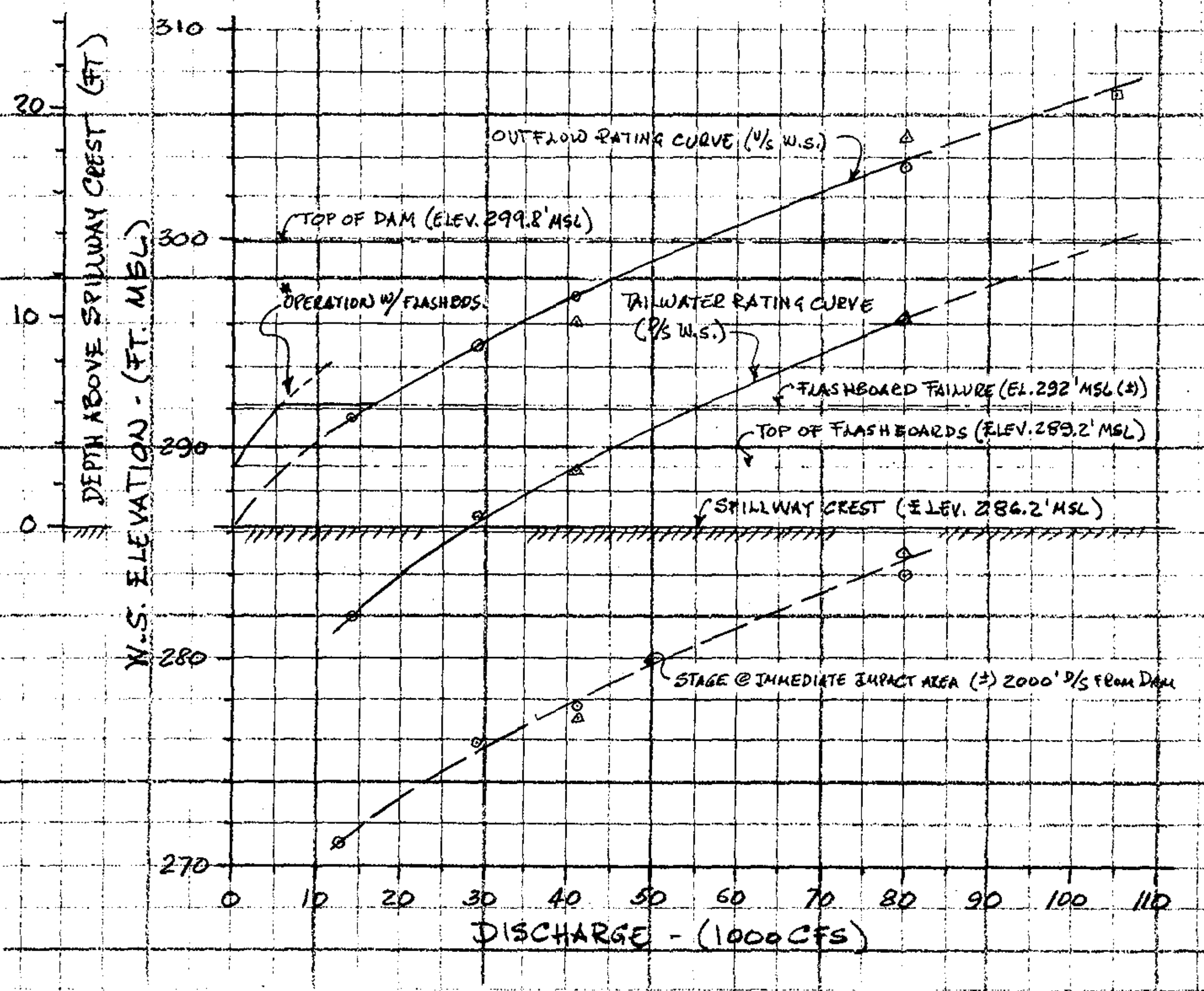
COLLINS CO. UPPER DAM

3.6 (Cont'd) OUTFLOW RATING CURVE



*NOTE: 3' FLASHBOARDS OVER THE ENTIRE LENGTH OF THE SPILLWAY ((±)L=330') ARE DESIGNED TO FAIL AT A HEAD OF 2' TO 3'. THIS OPERATION WILL NOT HAVE ANY EFFECT ON CONDITIONS AT TEST FLOOD.

△ - ACE

⊙ - HUD

⊡ - AUG. 1955

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Project NON-FEDERAL DAMS INSPECTION
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COLLINS CO. UPPER DAM

3-CONT'D) SURCHARGE AT PEAK INFLOW

c) SPILLWAY CAPACITY TO TOP OF DAM

$H \approx 14'$ (ELEV. 299.8' MSL) FROM CURVE $Q_S \approx 55000^{CFS}$
$((\pm) 66\%$ OF $Q_{P_1} = Q_{P_3})$

SPILLWAY SUBMERGENCE BY TAILWATER OF $(\pm)\ H_{TW} = 6'$

d) SURCHARGE HEIGHT TO PASS (Q_{P_1}):

$Q_{P_1} =$ "$\frac{1}{2}$ PMF" $\approx 83000^{CFS}$ $\therefore$ $H_1 \approx 18'$ SUBM.: $H_{TW} \approx 10.5'$

4) EFFECT OF SURCHARGE STORAGE ON MAX. PROBABLE DISCHARGES (OUTFLOW):

a) POND (LAKE) AVG AREA WITHIN EXPECTED SURCHARGE: $\bar{A} \approx 86^{AC}$ (see p. D-7)

b) WATERSHED AREA: D.A. = $359^{SQ\,MI}$ (see p. D-1)

c) PEAK OUTFLOW (Q_{P_3})

BECAUSE THE RESERVOIR STORAGE AT THE EXPECTED MAX. SURCHARGE (18') CORRESPONDS TO LESS THAN 0.1" R.O. OVER THE WATERSHED, NO APPRECIABLE PEAK REDUCTION IS PRODUCED BY SURCHARGE STORAGE AND,

$Q_{P_3} \approx Q_{P_1} = 83000^{CFS}$ $H_3 \approx 18'$ $(H_{TW} \approx 10.5')$ FOR $Q_{P_1} =$ "$\frac{1}{2}$ PMF"

d) SPILLWAY CAPACITY RATIO TO OUTFLOW:

THE SPWY CAP. RATIO TO (Q_{P_3}) IS THEREFORE $(\pm)$ THE SAME AS TO (Q_{P_1}) OR $(\pm)$ 66%
(see 3.c ABOVE)

COLLINS CO. UPPER DAM

I-(Cont'd) PERFORMANCE AT TEST FLOOD CONDITIONS

5) SUMMARY:

a) PEAK INFLOW: $Q_{P_1} = \frac{1}{2}$ PMF $\doteq$ 83000 CFS

b) PEAK OUTFLOW: $Q_{P_3} = Q_{P_1} \doteq$ 83000 CFS

c) SPILLWAY MAX. CAPACITY: $Q_S \doteq$ 55000 CFS OR, (±) 66% OF $Q_{P_1} = Q_{P_3}$

THEREFORE, AT SDF = ½ PMF, THE DAM IS OVERTOPPED (±) 4' ((±) W.S. EL. 304' MSL) OR TO A SURCHARGE OF (±) 18' ABOVE THE SPILLWAY CREST ELEV. 286.2' MSL. THE SPILLWAY WILL OPERATE UNDER SUBMERGED CONDITIONS IMPOSED BY A TAILWATER STAGE AT (±) W.S. ELEV. 296.5' MSL OR, (±) 10.5' ABOVE THE SPILLWAY CREST.

NOTE: BECAUSE CONDITIONS AT A TEST FLOOD = PMF WILL REQUIRE ANALYSIS BEYOND A PHASE I INVESTIGATION AND EXTRAPOLATION OF THE OUTFLOW RATING CURVE ON P. D-10 IS NOT WARRANTED, PARALLEL COMPUTATIONS AT SDF = PMF WILL NOT BE MADE FOR THIS PROJECT.

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
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Date 7/20/79
Revisions

COLLINS CO. UPPER DAM

II) DOWNSTREAM FAILURE HAZARD

1) PEAK FLOOD AND STAGE AT IMMEDIATE IMPACT AREA:

a) FAILURE CONDITIONS OF DAM:

i) IF FAILURE IS ASSUMED TO OCCUR WHEN THE SURCHARGE IS TO THE TOP OF THE DAM, THE RIVER AT THE IMPACT AREA ((±) 2000' D/S FROM THE DAM) IS AT A STAGE OF (±)*281' MSL PRODUCED BY THE FLOW OVER THE SPILLWAY JUST BEFORE FAILURE (Q_S ≐ 55000 CFS SEE P. D-11). THE TAILWATER IS SUBMERGING THE SPILLWAY AND THE SURCHARGE U/S FROM THE DAM IS (±) 8' ABOVE THE TW. THEREFORE, UPON FAILURE OF THE DAM ON THESE CONDITIONS, A RAISE IN STAGE* OF APPROX. 1' (SAY TO WL. ELEV. (±) 282' MSL) IS EXPECTED AT THE IMMEDIATE IMPACT AREA WHICH IS ALREADY INUNDATED BY THE HIGH FLOOD STAGE IN THE CHANNEL. (*SEE NOTE ON P. D-15)

ii) FAILURE WHEN THE WL. AT THE DAM IS AT SPILLWAY CREST ELEV. 286.2' MSL AND THE TAILWATER IS LOW ((±) WS ELEV. *267' MSL) IS A CONDITION WHICH MAY RESULT IN CRITICAL FLOODING AND, THEREFORE, WILL BE ANALYZED.

b) ANALYSIS FOR CONDITION (a, ii)

i) MID-HEIGHT (±) ELEV. 277' MSL. (MID HT BELOW SPWY CREST EL. 286.2' MSL)
- SPILLWAY HEIGHT (SEE P. D-7) IS H ≐ 18'

*NOTE: BY INTERPOLATION FROM STAGES AT THIS LOCATION FOR VARIOUS FLOWS, FROM W.S. PROFILES IN THE NED-ACE AND HUD-FIA FLOOD PLAIN INFORMATION / FLOOD INSURANCE REPORTS. (SEE PP. D-9 AND D-10)

COLLINS CO. UPPER DAM

1,b-Cont'd) - ANALYSIS FOR CONDITION (a,ii)

ii) APPROX. MID-HEIGHT LENGTH: $L \simeq 330'$

∴ iii) BREACH WIDTH (SEE NED-ACE D/S DAM FAILURE GUIDELINES):

$$W = 0.4 \times 330 = 132' \quad \therefore \text{ ASSUME } W_b = \underline{\underline{130'}}$$

iv) HEIGHT AT TIME OF FAILURE: $Y_0 = 18'$

v) BREACH OUTFLOW (Q_b):

$$Q_b = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2} \simeq 16700^{\text{CFS}}$$

vi) PEAK FAILURE OUTFLOW:

$$Q_{P_1} \simeq Q_b \simeq \underline{\underline{16700}}^{\text{CFS}}$$ (NO SPWY DISCH. AND POWER CO. TUR-BINE DISCH. ASSUMED NEGLIGEABLE)

vii) FLOOD DEPTH IMMEDIATELY D/S FROM DAM:

$$Y \simeq 0.44\, Y_0 \simeq 7.9' \quad \underline{\text{SAY, } \underline{8'}}$$

viii) FLOOD STAGE AT IMMEDIATE IMPACT AREA: $W.S.EL \simeq \underline{\underline{273'}}$ MSL
(± BY INTERPOLATION ON CURVE p. D-10)

ix) RAISE IN STAGE AFTER FAILURE: $\Delta Y \simeq 273 - 267 \simeq \underline{\underline{6'}}$

NOTE: FOR THIS CONDITION, THE IMPACT AREA ALSO IS ASSUMED (±) 2000' D/S FROM THE DAM, FOR COMPARATIVE/COMPUTATIONAL PURPOSES. BEING THE FARMINGTON RIVER RECREATIONAL USE THE PRIMORDIAL POTENTIAL HAZARD, THIS IMPACT AREA LOCATION IS ARBITRARY.

COLLINS CO. UPPER DAM

II-CONT'D) DOWNSTREAM FAILURE HAZARD

2) SUMMARY:

a) FAILURE W/ SURCHARGE TO TOP OF DAM:

i) SPILLWAY (SUBMERGED) DISCHARGE BEFORE FAILURE: $Q_S \doteq 55000^{cfs}$

ii) STAGE AT IMMEDIATE IMPACT AREA UPON FAILURE OF THE DAM, IS ESTIMATED TO RAISE* APPROX. 1' FROM (±) ELEV. 281' MSL TO (±) ELEV. 282' MSL.

b) FAILURE W/ SURCHARGE TO SPILLWAY CREST:

i) PEAK FAILURE OUTFLOW: $Q_{P_1} \doteq 16700^{cfs}$

ii) FLOOD DEPTH IMMEDIATELY D/S FROM DAM: $Y \doteq 8'$

iii) APPROXIMATE STAGE BEFORE FAILURE AT ASSUMED IMMEDIATE IMPACT AREA (±) 2000' D/S - SAME AS FOR CONDITION (a)): WS EL. $\doteq$ 267' MSL

iv) APPROX. STAGE AFTER FAILURE AT ASSUMED IMMEDIATE IMPACT AREA: W.S. EL $\doteq$ 273' MSL

v) RAISE IN STAGE AT ASSUMED IMPACT AREA: $\delta Y \doteq 6'$

*NOTE: A SIMILAR BREACH ANALYSIS TO THAT ON P. D-14 FOR CONDITION (a, ii), GIVES FOR POOL AT TOP OF DAM A FAILURE DISCHARGE $Q_P \doteq Q_S + Q_F \doteq 60000^{cfs}$ ($Q_F \doteq 5000^{cfs}$; $h_o = 8'$). THEREFORE, THE CORRESPONDING RAISE IN STAGE AT THE IMMED. IMPACT AREA IS (±) 1' OR, TO EL. 282' MSL. (SEE P. D-10)

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

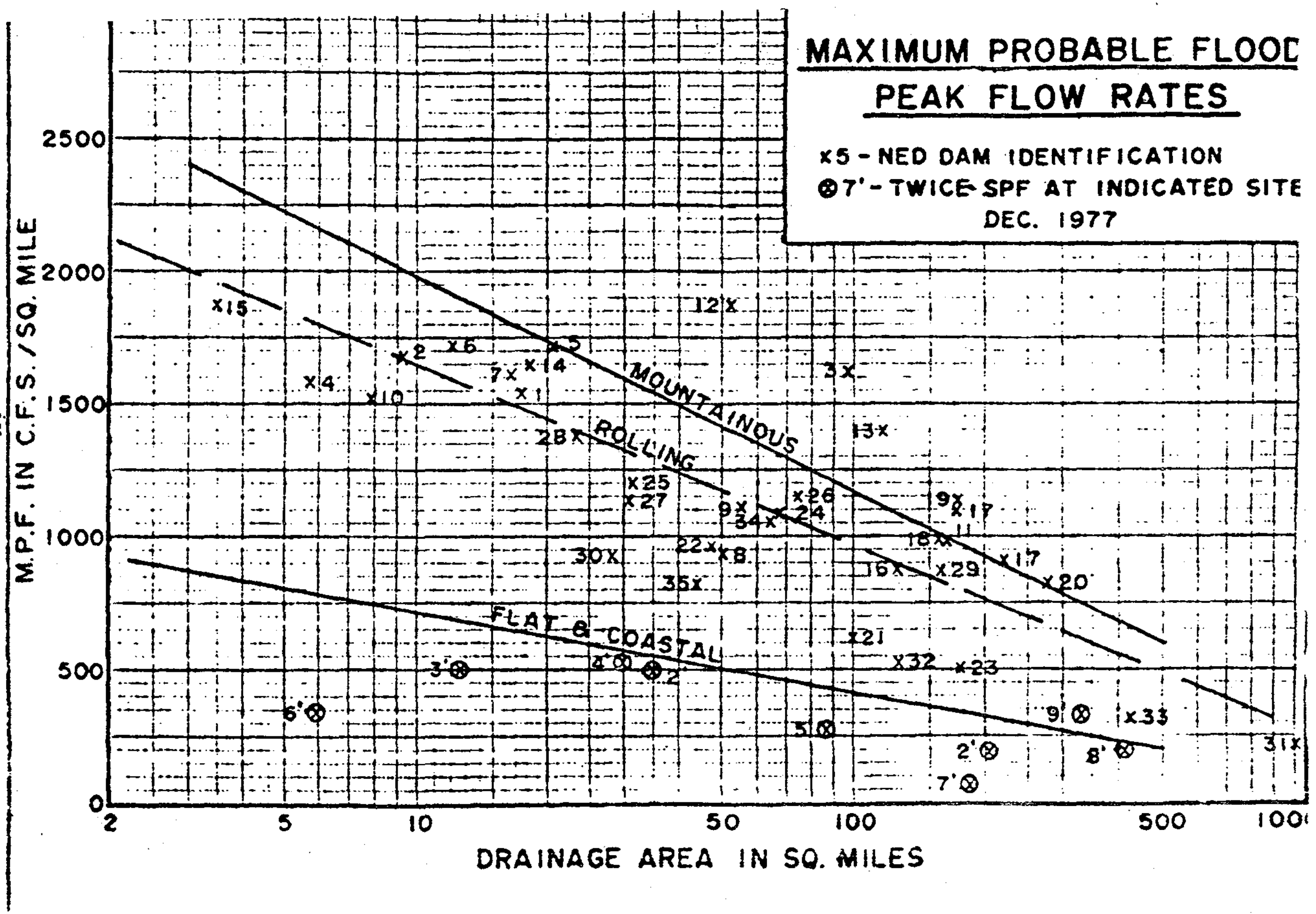
March 1978

MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

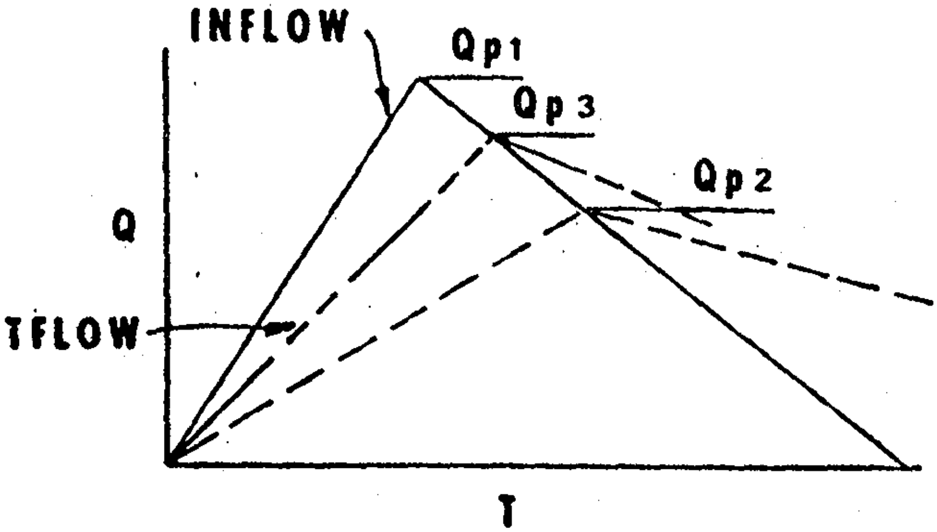
	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS BASED ON TWICE THE STANDARD PROJECT FLOOD (Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330


MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITE
DEC. 1977
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
MOUNTAINOUS
ROLLING
FLAT & COASTAL
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Q_{p1}".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Q_{p2}"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Q_{p3}".

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

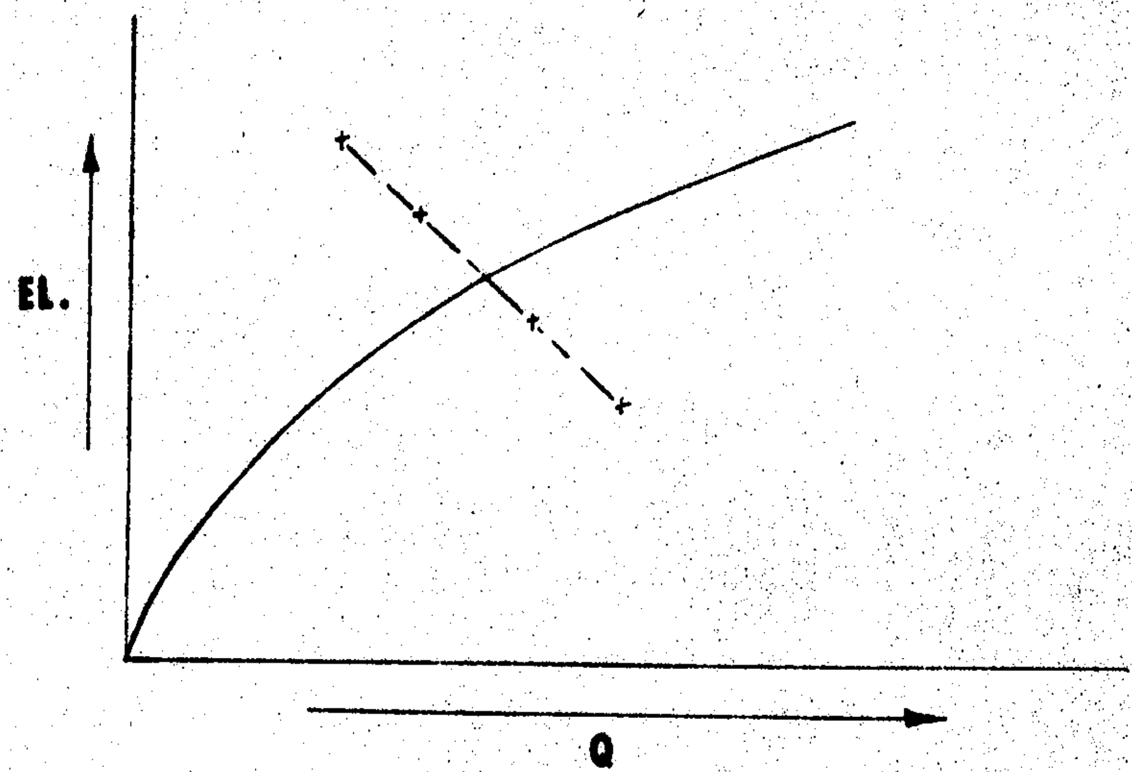
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

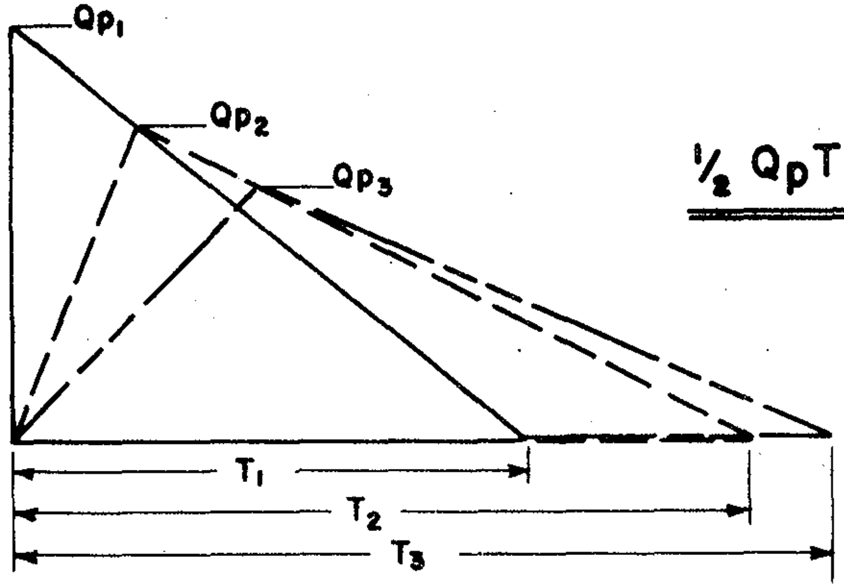
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19'' R.O.

Q_{p2}	STOR	EL.



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\frac{1}{2} Q_p T = 12\ S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27}\ W_b \sqrt{g}\ Y_o^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{ave}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS